REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
1. REPORT NUMBER Technical Report S-78-3	2. GOVT ACCESSION NO.	3. RECIPIENT'S CATALOG NUMBER
4. TITLE (and Subtitle) DAMAGE TO DAMS FROM NUCLEAR WEAPONS EFFECTS (FORMERLY ESSEX II); VULNERABILITY OF AN EARTH DAM TO BURIED EXPLOSIONS, PRELIMINARY STUDIES		5. TYPE OF REPORT & PERIOD COVERED Final report 6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) Walter C. Sherman, Jr.		8. CONTRACT OR GRANT NUMBER(s)
9. PERFORMING ORGANIZATION NAME AND ADDRESS U. S. Army Engineer Waterways Experiment Station Geotechnical Laboratory P. O. Box 631, Vicksburg, Miss. 39180		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS See Block 18
11. CONTROLLING OFFICE NAME AND ADDRESS Director Defense Nuclear Agency Washington, D. C. 20305		12. REPORT DATE May 1978 13. NUMBER OF PAGES 60
14. MONITORING AGENCY NAME & ADDRESS(if different from Controlling Office)		15. SECURITY CLASS. (of this report) Unclassified 15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

Distribution limited to U. S. Government agencies; test and evaluation; May 1978. Other requests for this document must be referred to the Director, Defense Nuclear Agency, Washington, D. C. 20305.

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES

This research was sponsored by the Defense Nuclear Agency under Subtask L19GAXYX979, Work Unit 03, "Erosion Failure Studies."

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

Dam failures
Dam foundations
Earth dams
Earth dam seepage
Nuclear weapons effects
Sliding
Slope failures
Underground explosions
Vulnerability

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The possibilities of deliberately failing an earth dam or causing such damage as to require immediate and extensive emergency repair measures to prevent total collapse or failure were explored. The mechanisms by which total or partial failure is induced as a result of buried explosions at critical locations within or in the vicinity of a typical earth dam were investigated.

The study was limited to a typical earth dam section with variable material properties and with variable foundation conditions. Various modes of failure were investigated involving: (1) uncontrolled embankment seepage, (2) uncontrolled seepage through the foundation, and (3) failure of the embankment slope by sliding. The effects of large explosions that would induce failure by overtopping were not considered.

(Continued)

20. ABSTRACT (Continued).

The basis for evaluating the mechanics of failure of an earth dam due to a buried explosion was reviewed and analyzed. The literature shows that the principal cause of catastrophic failure was overtopping. Other causes were shown to be progressive erosion and earth slides in the downstream portion of the dam and foundation.

The study reported herein was based on consideration of a hypothetical earth dam consisting of compacted earth with a central impervious core and pervious shells. The dam was 150 feet high and both the upstream and downstream slopes were 1V on 3H. The crest width was 20 feet and the reservoir pool height was assumed to be 120 feet. A number of variable factors were introduced in the study to simulate possible variations in design, including thickness, permeability, and shear strength of the foundation strata and various embankment seepage and foundation underseepage control measures. Seepage control measures within the embankment included a chimney drain and downstream drainage blanket. It was assumed that the outer shells were several times more permeable than the core so that seepage into the downstream shell was well controlled. Underseepage control measures included a central impervious cutoff, an upstream blanket, and relief wells along the downstream toe.

Available data are insufficient to estimate reliably the size and characteristics of camouflet that would be formed by a buried explosion. Practically nothing is known concerning the nature of the rupture zone and extent of cracking which might develop in both saturated and unsaturated materials as might be involved in an earth dam. The magnitude and distribution of peak and residual pore pressures which would develop are unknown. Further experimental data are needed in these areas.

In the case of the model dam selected for the study, it is considered quite probable that detonation of a 20-ton buried explosion located so as to perforate the central core will cause sufficient cracking and internal erosion, depending on the nature of the embankment materials, as to result in breaching of the dam. A small-scale model test indicated that a subsidence crater would form upstream of the core after detonation and that seepage would be somewhat greater than that for a homogeneous dam. Breaching of the dam, should it occur, would probably take place in less than 5 hours once significant seepage water appeared on the downstream embankment. Buried explosions in the foundation of the dam or beneath the downstream slope would be less effective in producing failure than explosions of similar yield within the embankment itself.

Although the study concerns the effects of a buried explosion on a typical earth dam, it must be emphasized that every dam is unique in cross section, and the embankment slopes, zonation, and internal drainage details are tailored to the specific site conditions, the materials available for construction, and other factors. The behavior of a dam cannot always be reliably predicted during its design life where all of the details are reasonably known during the design of the structure. It is much more difficult to assess the probable behavior of a dam under the action of a buried explosion for which practically no experience is available. Small-scale tests, particularly if tested in a centrifuge, appear to offer the best means for more detailed study. Three-dimensional electrical analogy tests would also provide useful information. An exceedingly important question concerns the nature and extent of cracking generated by the blast in various soil types and the effects of cracking on possible piping failure. The consequences of cracking and methods for its control are subjects of much current interest to designers of earth dams. Development of a laboratory research program to evaluate piping resistance of cracked embankment material would be worthwhile in this regard.

PREFACE

The study of vulnerability of dams reported herein was conducted under the sponsorship of the Defense Nuclear Agency and was performed as part of the Nuclear Weapons Effects Program, Subtask L19GAXYX979, Work Unit 03, "Erosion Failure Studies." The work was accomplished during the period December 1975 through March 1976 by Mr. W. C. Sherman, Jr., under the direction of Mr. J. P. Sale, Chief, Geotechnical Laboratory. Defense Nuclear Agency Subtask Manager was MAJ T. D. Stong, U. S. Army. This report was written by Mr. Sherman.

COL G. H. Hilt, CE, and COL J. L. Cannon, CE, were Directors of the U. S. Army Engineer Waterways Experiment Station during the study and publication of this report. Mr. F. R. Brown was Technical Director.

CONTENTS

LIST OF ILLUSTRATIONS

LIST OF TABLES

CONVERSION FACTORS, U. S. CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

U. S. customary units of measurement used in this report can be converted to metric (SI) units as follows:

Multiply	By	To Obtain
inches	2.54	centimetres
feet	0.3048	metres
feet per minute	0.00508	metres per second
pounds (mass)	0.4535924	kilograms
tons (2000 pounds)	907.1847	kilograms
pounds per square foot	4.882428	kilograms per square metre
gallons (U. S. liquid)	0.003785412	cubic metres

DAMAGE TO DAMS FROM NUCLEAR WEAPONS EFFECTS

Preliminary Studies

Vulnerability of an Earth Dam to Buried Explosions

CHAPTER 1

INTRODUCTION

1.1 OBJECTIVE

The objective of this report was to explore the possibilities of deliberately failing an earth dam or causing such damage as to require immediate and extensive emergency repair measures to prevent total collapse or failure. For this purpose, the mechanisms by which total or partial failure is induced as a result of buried explosions at critical locations within or in the vicinity of a typical earth dam were investigated. It was planned to establish, if possible, relationships between explosive yield and degree of damage, with time to failure considered an important factor.

1.2 SCOPE

This study was limited to a typical earth dam section with variable material properties and with variable foundation conditions. Various modes of failure were investigated involving: (1) uncontrolled embankment seepage, (2) uncontrolled seepage through the foundation, and (3) failure of the embankment slope by sliding. The effects of large explosions that would induce failure by overtopping were not considered. Brief comments are provided concerning vulnerability of hydraulic-fill dams and rock-fill dams.

CHAPTER 2

APPROACH

2.1 CAUSES OF EARTH DAM FAILURES

The basis for evaluating the mechanics of failure of an earth dam due to a buried explosion is best determined by a review and analysis of actual failures that have occurred. Middlebrooks[1] made a survey of 206 earth dam failures which had occurred during the period from 1850 to 1952 and showed that the principal cause of catastrophic failure was overtopping. Other causes of catastrophic failures were shown to be progressive erosion (piping) and earth slides in the downstream portion of the dam and foundation. Goldsmith[2] made a similar study of earth and rock-fill dam failures for the period 1950 to 1972. A total of 105 failures were reported. The review indicated conclusions similar to those of the study by Middlebrooks.

The meaning of the word "failure" requires careful consideration. As pointed out by Babb and Mermel,[3] the number of reported failures can statistically be very misleading as failures falling within the popular concept of a catastrophic disaster have been comparatively few. These authors suggest classification of failures into the following groups:

a. Major disasters. The sudden and complete failure of a dam while in service, usually with total destruction and loss of the dam and of life and property.

b. Failures and washouts of minor dams constructed without benefit of professional skill and having little engineering significance.

c. Failures to gates, valves, piers, and appurtenant structures including cracks, leaks, and erosion where the dam as such did not fail but received publicity associated with its name.

d. Accidents and failures during construction and before the dam was ready for service.

e. Failures to dams built prior to the modern era of dam building about which little is known.

A comprehensive survey of dam incidents throughout the world was conducted by the International Commission on Large Dams and the results were published in a reduced edition[4] in 1973. The survey of dam

incidents in the United States conducted by the United States Committee on Large Dams[5] is particularly noteworthy in providing an insight into the frequency and degree of damage. Incidents were categorized as shown in Table 2.1. Reference 5 clearly indicates that properly designed, constructed, and maintained dams are safe structures. The percentage of subsequently failed dams constructed in any decade has generally decreased decade by decade. Failures caused by overtopping have been practically eliminated in dams completed after 1930, although it must be noted that most of these dams have not yet experienced their maximum reservoir pools. Leakage in the foundation and embankment of earth and rock-fill dams was the most frequent cause of failures and accidents.

Casagrande[6] has summarized the various causes and modes of failures of earth dams as shown in Figure 2.1. Experience indicates that the three main requirements to prevent failure of earth and rock-fill dams, excluding requirements to prevent overtopping, are: (1) the dam must be safe with respect to piping due to uncontrolled seepage within the embankment, (2) the dam must be safe with respect to piping due to uncontrolled underseepage in the foundation, and (3) the slopes of the dam must be safe with respect to sliding. Various defensive measures are incorporated in the design to satisfy all of these requirements. Excellent summaries of design requirements are contained in References 7 and 8. A review of the critical defensive measures incorporated to satisfy the above requirements provides a means for assessing the potential for damage induced by buried explosions.

2.2 HYPOTHETICAL DAM

The approach to the study was based on consideration of a hypothetical earth dam. The model dam consists of a compacted earth dam with central impervious core and pervious shells (see Figure 2.2). The height is 150 feet* with both the upstream and downstream slopes of 1V on 3H. The crest width is 20 feet and the reservoir pool height is assumed to

---

* A table of factors for converting U. S. customary units of measurement to metric (SI) units is presented on page 4.

Table 2.1. Categories of dam incidents.

Definitions employed by the Committee on Failures and Accidents to Large Dams of the United States Committee on Large Dams.

Category	Definitions
A	Failure, Type 1 - A major failure of an operating dam which has involved complete abandonment of the dam. (Information on downstream effects of the dam failure should be included if pertinent.)
B	Failure, Type 2 - A failure of an operating dam which at the time may have been severe, but was of a nature and extent which permitted the damage to be successfully repaired and the dam again placed in operation.
C	Accident, Type 1 - An accident to a dam which had been in operation for some time, but which was prevented from becoming a failure by remedial work or operations such as drawing down the pool.
D	Accident, Type 2 - An accident to a dam observed during the initial filling of the reservoir, which caused immediate remedial measures, including such action as drawing down the water level and making repairs before placing the dam in operation.
E	Accident, Type 3 - An accident to a dam before it was placed in operation and before any water was impounded. Unusual settlement of a foundation, slumping and slides of the abutments, etc., after essential completion of the structure would be accidents of this type. It is not intended to include the normal construction problems and movements which occur during construction operations.
F	Accident, Reservoir - Accidents or unusual problems encountered in the reservoir upstream of the dam which have occurred during operation of the project, but which have not caused failure or major accident to the dam structure.
G	Damage During Construction - Damage to partially constructed dam or to temporary structure required for construction prior to the dam being essentially completed. Failure of cofferdam or unplanned overtopping of partially completed dam are examples.
H	Major Repair - Extensive or important repairs to a dam that were required because of deterioration or to update certain features. Refacing of deteriorated concrete, repair of deteriorated riprap, or replacement of gates are examples.

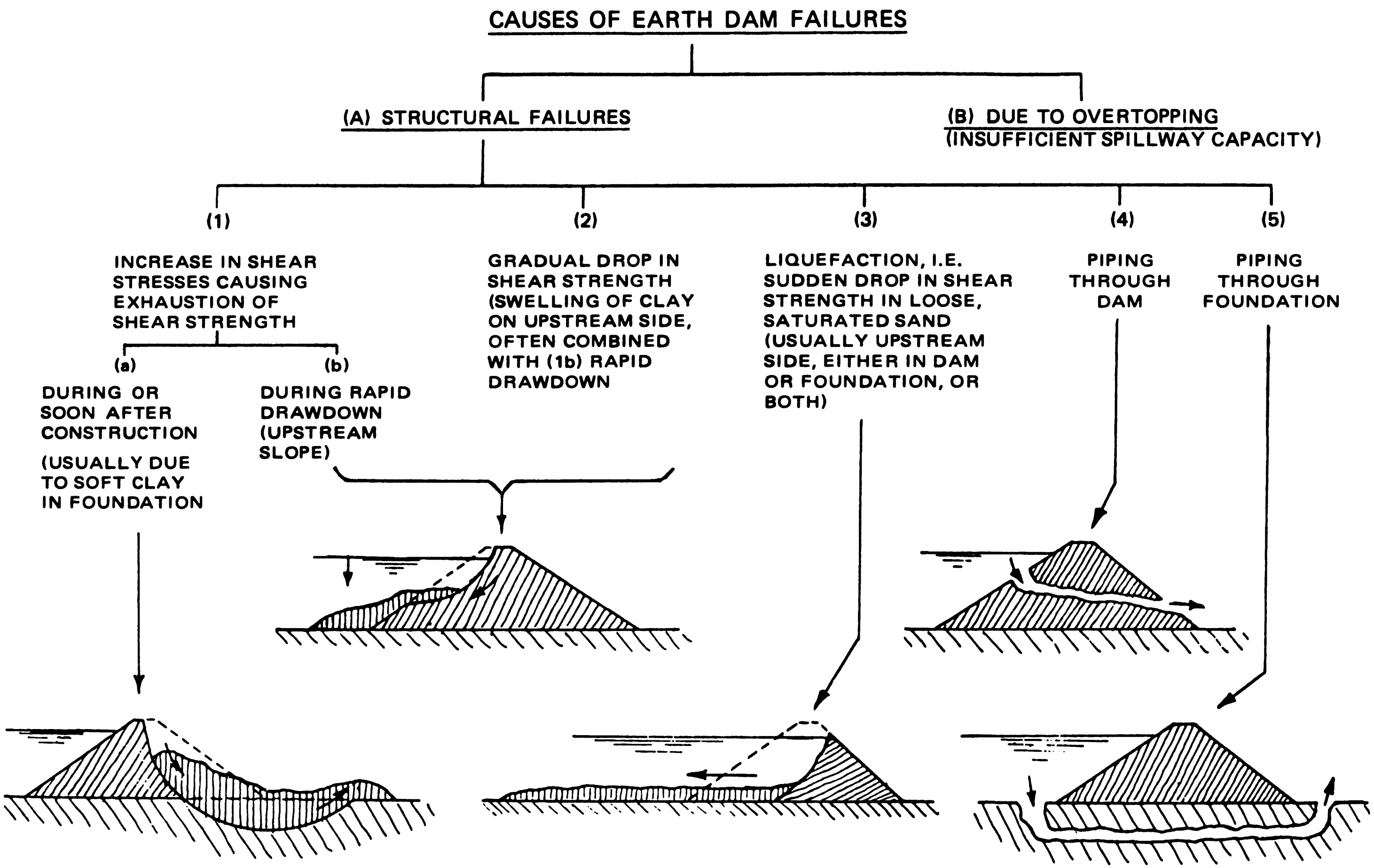


Figure 2.1. Causes of earth dam failures.

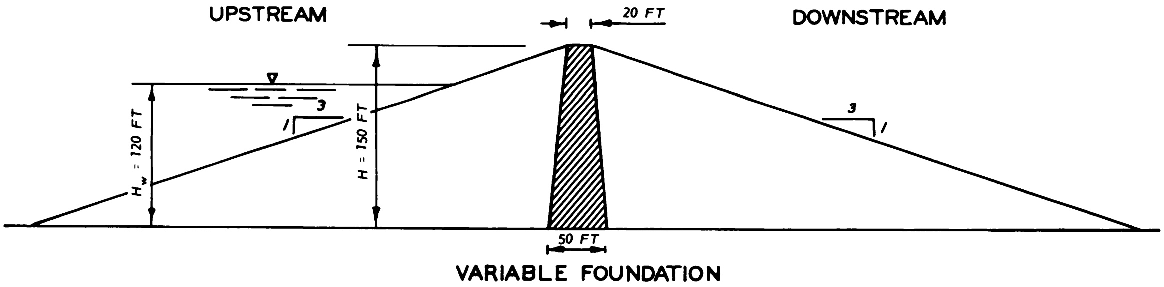


Figure 2.2. Hypothetical earth dam.

be 120 feet. This basic section was used in all analyses.

Selection of the model dam dimensions presupposed certain soil constants and properties which provide the design basis for the embankment slopes. Further, the design details of any dam within the geometric limits selected for the model will vary depending, in particular, on foundation conditions and soil types available for construction of the embankment. Consequently, a number of variable factors were introduced in the study to simulate possible variations in design. The variable factors included the thickness, permeability, and shear strength of the foundation strata and various embankment seepage and foundation underseepage control measures. Seepage control measures within the embankment included a chimney drain and downstream drainage blanket. It was assumed that the outer shells were several times more permeable than the core so that seepage into the downstream shell was well controlled. Underseepage control measures included a central impervious cutoff, an upstream blanket, and relief wells along the downstream toe. Of critical importance in any dam are the details of design, particularly for the drains and transition zones and drainage provisions that will have a marked influence on the behavior of the dam when subjected to severe dynamic loading and associated cracking. These details would have to be taken into account in an assessment of the vulnerability of a specified dam, but they have not been taken into consideration in the subject study.

2.3 EXAMPLE OF EXPLOSIONS ON DAMS

The only known example of an earth or rockfill dam subjected to large explosive forces is the case of Sorpe Dam. This dam in the Ruhr

District, Federal Republic of Germany, was subjected to two bomb attacks during World War II.[9] The dam is about 200 feet high and was completed in 1934. It was constructed of earth and rock fill with a central concrete core wall. The dam was first attacked in May 1943 and received two hits at the upstream side that did not penetrate the crest of the dam. However, the clay sealing of the dam was damaged and had to be restored immediately. Seepage through the concrete core increased by 30 to 50 percent. In October 1944, a second bomb attack took place. The dam got nine hits forming craters with depths to 12 metres and diameters of 25 to 30 metres. Despite considerable damage at the crest, no water broke through the dam as the water level in the reservoir had been lowered below storage level. Although the bomb craters were immediately filled up and the clay sealing was restored as well as possible, the amount of seepage water increased by as much as 100 percent. There was evidence that numerous cracks and fissures had developed within the dam as a result of the explosions. In the beginning of 1951, a large increase in seepage occurred in the drainage system and in the inspection galleries. Sinkholes reaching from the dam surface down to the lower drainage system developed, and the embankment rapidly settled by 1.4 metres. A general restoration of the dam was subsequently made.

2.4 EFFECTS OF BURIED EXPLOSIONS

The effects of buried explosives are illustrated in Figure 2.3. Charges detonated at the optimum depth of burst (DOB) will produce the largest surface crater in terms of volume. At depths greater than optimum DOB, the effects of buried explosions will vary widely as shown in Figure 2.3b, 2.3c, and 2.3d and are difficult to predict because of the scarcity of experimental data on a wide variety of foundation media. A summary of available data on the effects of buried explosions is contained in Reference 10. Of primary interest to the subject study are camouflets produced by buried explosions shown in Figure 2.3d. Available information on camouflets is presented in this chapter.

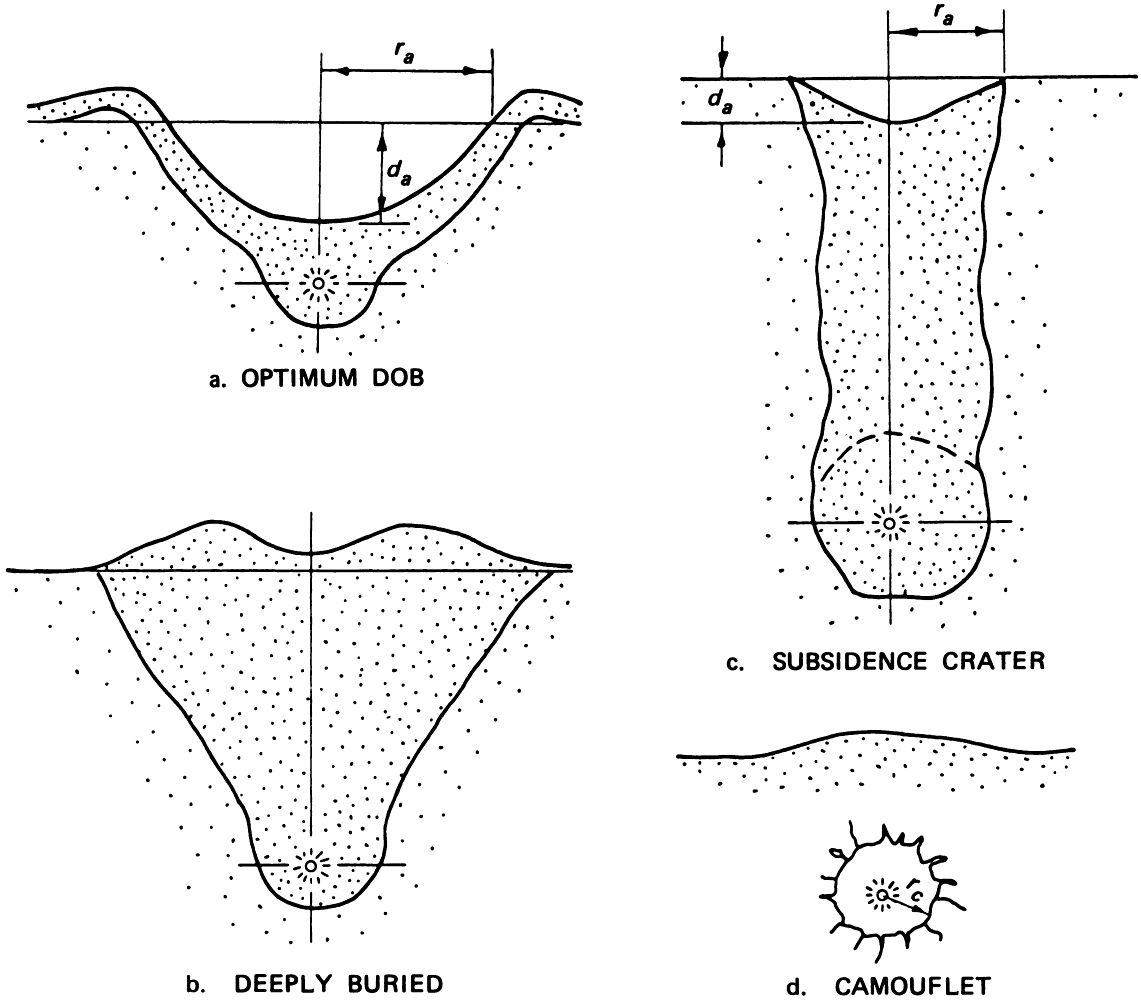


Figure 2.3. Results of buried explosions.

2.4.1 Size of Camouflet

A series of high explosive (HE) charges varying from 1/2 to 1 pound in weight were detonated in clay and loess above the water table producing camouflets in the range of 1 to 5 feet in diameter (Reference 11). It was found that the camouflets were, for practical purposes, symmetrical. The radius of the camouflet could be predicted as follows:

$$r_c \simeq 1.15\ W^{1/3} \tag{1}$$

where

r_c = average radius, feet

W = charge weight, pounds

The relation between r_c and W is shown as curve 1 in Figure 2.4 for charge weights between 20 and 100 tons. Presumably, considerable error is involved in extrapolation of the above-mentioned tests with small charges to much larger charges which might be used to demolish an embankment dam. Camouflets were produced when the charge was positioned below a depth Z_c given by the expression:

$$Z_c = \simeq 3.2W^{1/3} \tag{2}$$

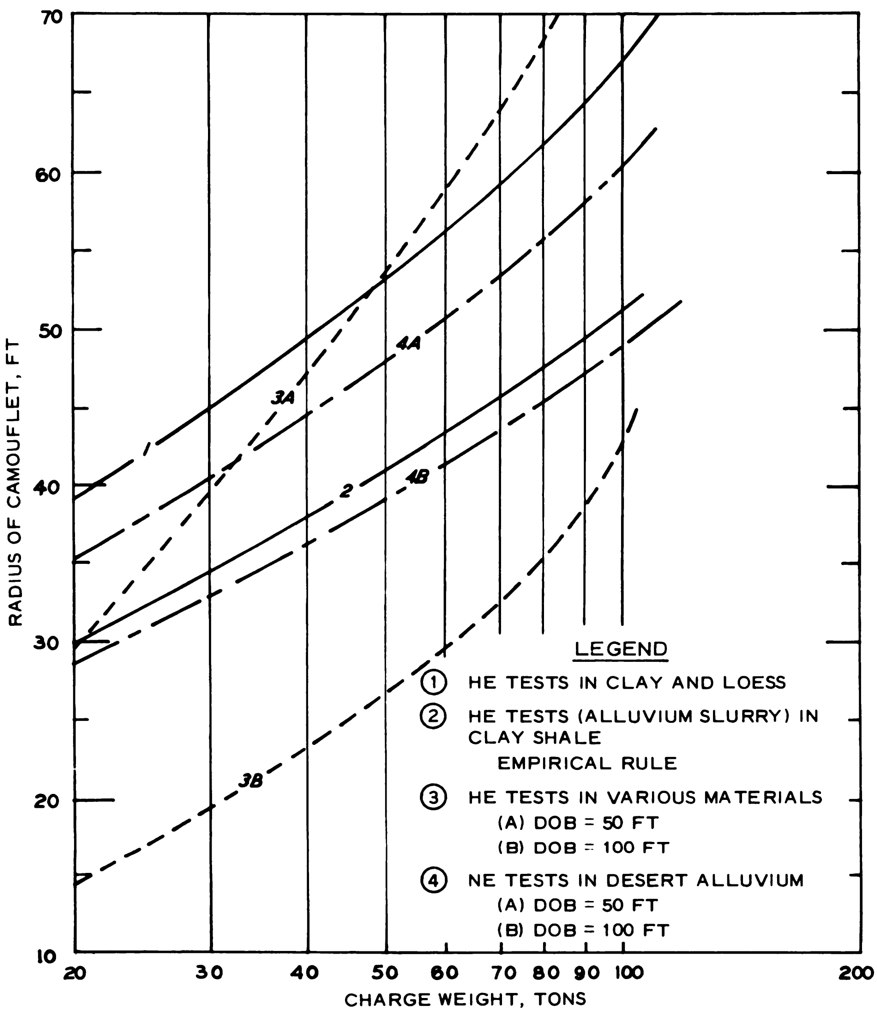


Figure 2.4. Charge weight versus radius of camouflet.

An empirical rule developed at WES for contained bursts of ammonium nitrate slurry in saturated clay shale indicates that the cavity volume will be approximately 200 times the explosive charge volume. The depth of burial must be greater than 7 $ft/lb^{1/3}$. Based on a slurry density of 1.12 g/cm^3, the calculated radius of camouflet for various charge weights is shown as curve 2 in Figure 2.4.

An expression for camouflet radius as a function of charge weight for HE tests in various materials is presented in Reference 10.

$$r_c = \frac{1.25W^{2/3}}{DOB} \qquad (3)$$

where

r_c = average radius, feet

W = charge weight, pounds

DOB = depth of burst, feet

The relations between r_c and W for DOB of 50 and 100 feet are shown as curves 3a and 3b, respectively, in Figure 2.4.

Reference 10 also presents an expression for camouflet radius produced by nuclear explosions in desert alluvium. The equation is

$$r_c = \frac{89W^{1/3}}{(\rho h)^{\alpha}} \qquad (4)$$

where

r_c = average radius, metres

W = charge weight, kilotons

ρ = density, g/cm^3

h = DOB, metres

α = constant taken as 0.296

The relations for camouflet radius as a function of charge weight calculated for $\rho = 1\ g/cm^3$ and depths of burial of 50 and 100 feet are shown as curves 4a and 4b, respectively, in Figure 2.4.

It is apparent from Figure 2.4 that the range of camouflet sizes from a buried explosion can vary over wide limits. The range of values is sufficiently large to justify further experimental testing. Most of

the test data apply to tests in relatively homogeneous nonsaturated media below a level ground surface. Such conditions are not realized within a zoned embankment which may be saturated upstream of the core and drained on the downstream side. From the dam in Figure 2.4, it has been assumed that camouflets of 30- and 50-foot radius can be produced by 20- and 100-ton yield detonations, respectively. It is also assumed that explosive devices of the above yield can be detonated at selected locations within the hypothetical dam or its foundation. The ideal yield is that having the minimum energy to accomplish the desired effects. For the hypothetical dam, only the 20-ton yield device was considered at the base of the core as larger devices would tend to form craters which would breach the crest.

2.4.2 Fractures

Although a camouflet of a size corresponding to the estimated design values may form initially, it would be immediately subjected to large seepage forces if detonated in a saturated soil and certainly collapse in cohesionless soils. In addition to creating cavities or craters, buried explosions will tend to induce a large amount of cracking in the vicinity of the detonation. Cratering experiments in rock indicated that the rupture zone will extend at least one radius beyond the cavity radius. Cracks will probably tend to be much larger and extend for greater distances as a result of hydraulic fracturing when the soil is submerged. It is often assumed that open cracks cannot exist in cohesionless materials such as sand, gravels, and sand-gravel mixtures. This may not always be the case, however, as recent work at the U. S. Army Engineer Waterways Experiment Station[12] (WES) suggests that cracks in cohesionless soils may stay open under the action of seepage forces.

2.4.3 Pore Pressure Effects

In addition to explosively produced cracks found in the immediate vicinity of the detonation, large pore pressures will be induced, particularly in saturated zones downstream of the core. The peak dynamic pore pressures will probably have little influence on the subsequent

behavior of the materials; however, the residual pore pressures may be quite high and have a significant influence on the materials' behavior. Buried explosions for the Pre-Gondola events in saturated clay shales resulted in substantial positive residual pore pressures. The magnitude and distribution of pore pressures induced by a buried explosion within the model dam are unknown and current experience does not permit even an approximate estimate. Pore pressures are of the utmost importance in the determination of liquefaction potential and in evaluation of stability of embankment slopes with respect to sliding.

2.5 SEEPAGE CHARACTERISTICS

Flow net analyses[13] offer only a crude means of establishing the pattern of flows in a dam disrupted by a buried explosion as the seepage pattern resulting from damage due to a buried explosion is three-dimensional in nature. A conventional two-dimensional flow net provides only an approximate indication of the flow pattern; however, more refined computational techniques are available for analyzing three-dimensional seepage problems. Flow nets indicate the points of seepage concentration where piping might develop and also permit an estimate of the amount of increased seepage that might develop as a result of the buried explosion. It should be recognized that volume of seepage is not a reliable index of damage, as a dam with a large volume of seepage and adequate control measures may be completely safe whereas a small volume of seepage with inadequate control measures can lead to severe damage or failure. Soil type is also an important factor.

While flow nets permit some estimate as to imposed damage, more important factors are the extent of cracking induced by the explosion and the piping resistance of the embankment materials. Unfortunately at the present time, no theoretical analyses are available for evaluating piping resistance. Only empirical data are available on which to judge the effects of the piping through such materials.

2.5.1 Soil Properties

The importance of soil properties in determining the resistance of

the dam to piping and progressive erosion due to concentrated leaks cannot be overemphasized.[14] Most piping failures have occurred on older and poorly constructed small dams. Current practice is to compact earth embankments with controlled water contents and density to minimize the development of concentrated leaks; the use of properly designed chimney drains downstream of the core also tends to control piping. A rough empirical relationship between piping resistance in earth dams and soil types and construction methods is shown in Table 2.2.

Table 2.2. Rough empirical relationship observed between piping resistance in earth dam embankments and soil types and construction methods (in order of decreasing piping resistance).

Piping Resistance	Soil Types and Construction Methods
Greatest	Clay of high plasticity (Plasticity Index (PI) greater than 15). Well compacted.
Greatest	Clay of high plasticity (PI greater than 15). Poorly compacted.
Intermediate	Well-graded coarse sand or sand-gravel mixtures with binder of clay of medium plasticity (PI greater than 6). Well compacted.
Intermediate	Well-graded coarse sand or sand-gravel mixtures with binder of clay of medium plasticity (PI greater than 6). Poorly compacted.
Intermediate	Well-graded cohesionless gravel-sand-silt mixtures (PI less than 6). Well compacted.
Least	Well-graded cohesionless gravel-sand-silt mixtures (PI less than 6). Poorly compacted.
Least	Very uniform fine cohesionless sand (PI less than 6). Well compacted.
Least	Very uniform fine cohesionless sand (PI less than 6). Poorly compacted.

2.5.2 Mechanics of Piping

According to Sherard and others,[7] the mechanics of piping are described as follows:

> As water seeps through the compacted soil of an embankment or the natural soil of a foundation, the pressure head is dissipated in overcoming the viscous drag forces which resist flow through the small soil pores. Conversely, the seeping water generates erosive forces which tend to pull the soil particles with it in its travel through and under the dam. If the forces resisting erosion are less than those which tend to cause it, the soil particles are washed away and piping commences. The resistant forces depend upon the cohesion and the interlocking effect and the weight of the soil particles as well as on the action on the downstream filter, if any.

CHAPTER 3

DAMAGE TO EMBANKMENT SEEPAGE CONTROL MEASURES

3.1 GENERAL CONSIDERATIONS

Although many dams have been seriously damaged as result of uncontrolled seepage within the embankment, examples of total failure or collapse have been relatively rare. Design measures used to control through seepage in modern earth dams generally consist of a horizontal drainage blanket and/or a chimney drain downstream of the core draining through the horizontal drainage blanket. These measures have been generally successful. Both experience and model tests indicate that failures as result of through seepage involve initiation of an unravelling failure along the lower portion of the slope. The resulting loss of material causes a crevice or channel which erodes headwards until increased gradients rapidly accelerate failure. A summary review of some of the embankment sections which have failed as a result of uncontrolled internal erosion or piping is shown in Table 3.1. The exact cause of the failure is generally indeterminate, and it is quite possible that the dams could have been subjected to seepage for a considerable time prior to actual failure. The time to failure listed in Table 3.1 generally refers to the time when significant quantities of seepage water were first noted emerging downstream of the dam. In numerous other cases, excessive leakage was observed, but time was available to lower the reservoir pool and apply corrective measures before failure occurred. All of the dam failures listed in Table 3.1, except for the Oxbow Project, were accidental with little or no advance warning before collapse occurred. The Oxbow Project involved deliberate failure of a fuseplug dam to determine the rate at which breaching would occur. It may be noted that the rate of failure, once appreciable quantities of visible seepage were noted, ranged from 0.05 to 3.4 ft/min, in terms of height of dam; in all cases, complete breaching occurred in less than 5 hours. Rates of failures in terms of base width of dam varied from 0.1 to 11.3 ft/min. The rate of failure obviously is influenced considerably by the type of soil and the type of seepage control measures incorporated in the dam.

Table 3.1. Rate of failure of dams subject to internal erosion.

Raised numbers refer to similarly numbered items in the References at the end of the main test.

Location	Material Type	Base Width (W), ft	Height (H), ft	Failure Time (t), hr	Rate of Failure, ft/min $\frac{W}{t}$	Rate of Failure, ft/min $\frac{H}{t}$
Baldwin Hills[15] Dam Abutment	Fine sand, silt, clayey silt, and clay	440	160	4.4	1.7	0.61
Hell Hole Dam Site[16]	Dumped rock fill	680	220	4.5	2.5	0.82
Earth Dam near Wheatland, Wyo.[17]	Unknown	185	45	1.5	2.1	0.50
Walter Bouldin[18] Dam[a]	Sandy clay	≃720	120	<5	>2.4	>0.40
Buffalo Creek[19]	Coal waste	≃400	≃50	3.5	≃1.9	≃0.24
Laguna Dam[20]	Silts and fat clays	135	30	4.75	0.5	0.11
Oxbow Project[21]	Uniform sand and gravel	--	13.5	0.25	--	0.90
Stockton Creek[22] Dam	Clayey sand	≃230	≃50	≃2	≃1.9	≃0.42
Teton Dam[23]	Zoned fill with silt core	1700	300	2.5	11.3	2.0
Hatchtown Dam[24]	Sandy silt	312	65	5	1.0	0.22
Dam A[25]	Lean clay	210	55	1.5	2.3	0.61
Dam B[25]	Unknown	≃70	15	5.6	0.1	0.05
Dam C[25]	Lean clay	241	51	(a) 15.5 (b) 3.5[b]	(a) 0.2 (b) 1.1	(a) 0.05 (b) 0.24
Dam D[25]	Unknown	234	25	4.6	0.8	0.08

[a]Failure initiated by upstream slide.
[b]Most of the breaching occurred during the last 3-1/2 hours.

The above discussion does not include piping failures in dispersive clays discussed by Sherard.[26] The failures generally involved relatively low homogeneous earth dams without filters and drains which would normally be incorporated in important dams of greater height.

3.2 EFFECTS OF BURIED EXPLOSIONS

It is obvious that the most strategic location of a buried explosion would be one that destroyed the central core and reduced its effectiveness as a seepage barrier. As noted by Sherard,[27] the top of the dam is particularly vulnerable because it is the thinnest part and has less internal pressure that tends to close the cracks. On the other hand, it may be more critical to detonate at greater depths within the embankment to produce a larger rupture zone involving more cracks and more leakage that would be difficult to control even though the reservoir should be lowered. In the following analyses, it is assumed that a 20-ton detonation near the base of the dam would produce a camouflet with a radius of 30 feet and a rupture zone on the order of a 60-foot radius or more. Detonation of a 100-ton device within the embankment would result in cratering that in turn would also involve surface breaching and overtopping. As these types of failures are the subject of other studies, the 100-ton device was not considered.

Upon detonation of the 20-ton device, the camouflet thus formed would most likely fill immediately with materials from the upstream shell as a result of seepage forces. The explosion would also tend to produce extensive cracking in the vicinity of the explosion that would extend to the slope surfaces. High pore pressures would be generated, particularly in the core and upstream shell materials, which might induce sliding of the upstream shell into the reservoir. The effects of the explosion on the pore pressures in the drained downstream shell would need to be investigated.

Complete rupture of the core as a result of the buried explosion would eliminate this element as a seepage barrier and the embankment would be subjected to steady seepage under the effect of the reservoir pool. To determine the exact pattern of seepage, the three-dimensional

aspects of the seepage pattern would have to be considered. Seepage would tend to channel through the core and dissipate laterally downstream of the core. This condition is amenable to theoretical analysis based on the finite element method. A rough estimate of the seepage can be made by means of flow nets in the section and half plan as shown in Figure 3.1.

To estimate the rate of seepage from a two-dimensional flow net of unit thickness, the following equation is used:

$$q = kh \frac{n_f}{n_c} \tag{5}$$

where

q = flow rate per unit width

k = coefficient of permeability

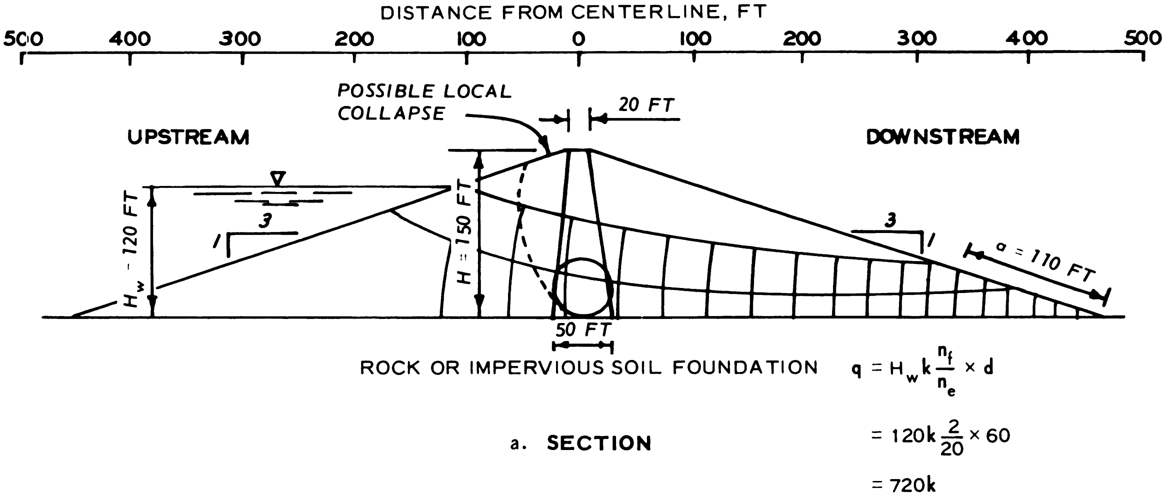
h = total head loss across the dam

n_f = number of flow channels

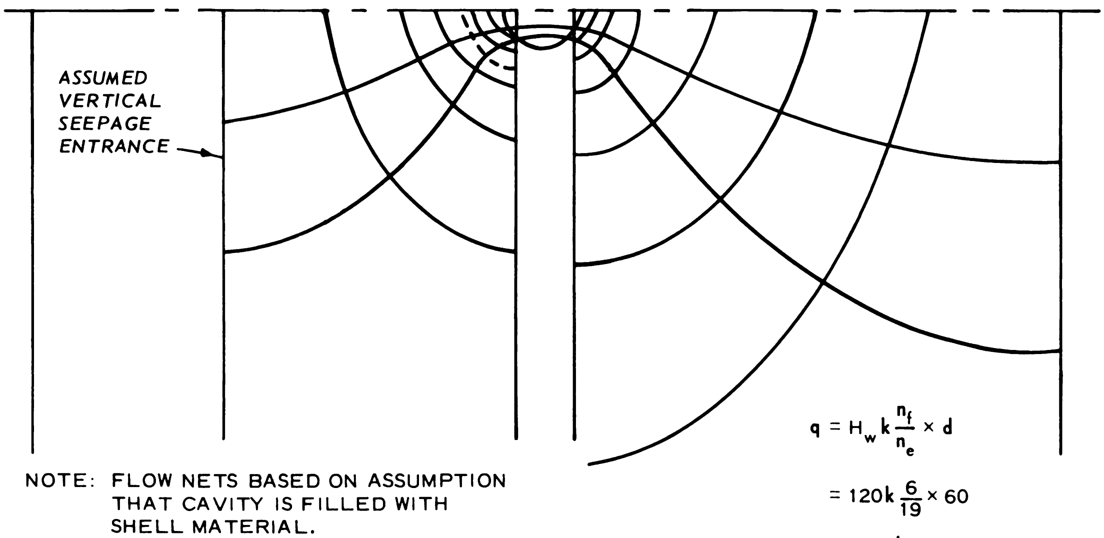
n_c = number of equipotential drops

The quantity of seepage for the model dam with a perforated core is computed to be equal to 12 k per foot of dam or a total of 720 k for the flow net in section and a total of 2280 k for the flow net in the half plan (see Figure 3.1). In drawing the flow nets, it was assumed that the camouflet would be filled with upstream shell material shortly after its formation.

In Figure 3.2 the amount of seepage is plotted as a function of type and permeability of shell materials. The seepage pattern through the embankment as indicated by the flow net is independent of the permeability; however, it would be affected by the ratio of the vertical to the horizontal permeability. Consequently, with very permeable shell materials, the amount of seepage can be quite large. However, as noted previously, the quantity of seepage in itself is no basis for evaluating whether the dam would be severely damaged. Consequently, the flow nets are of limited value in determining the severity of seepage with respect to possible damage. A more likely criteria are the extent of cracking and the types of materials as indicated in Table 2.2, with respect to



a. **SECTION**



b. **HALF PLAN**

Figure 3.1. Flow nets in section and plan for embankment with ruptured core.

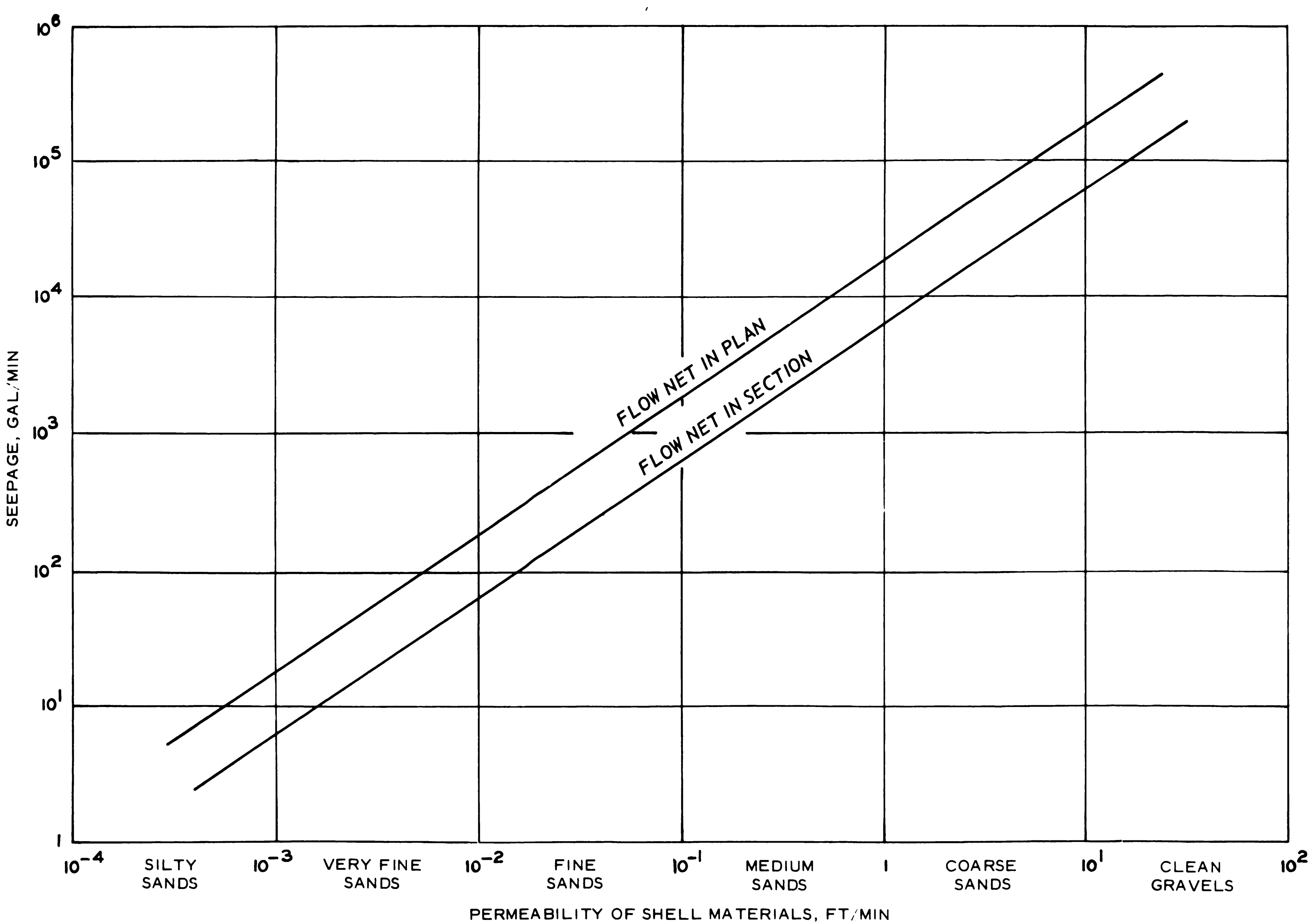


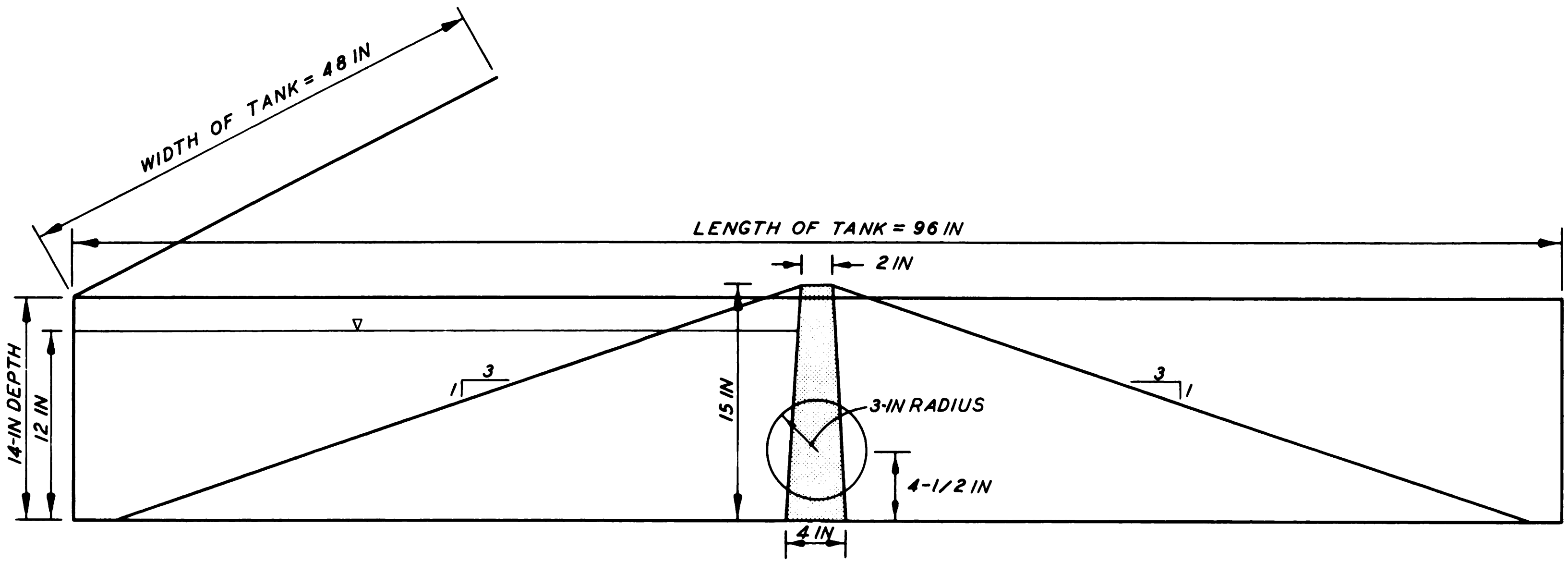
Figure 3.2. Additional seepage resulting from buried explosions.

the piping resistance. The principal factors which would determine whether the dam would fail as a result of piping are the extent of cracking, the types of materials in the downstream shell, and the provisions incorporated for controlling through seepage such as downstream filter blankets and toe drains. Thus, any determination of the probable effects would require detailed knowledge of these factors for a specific dam.

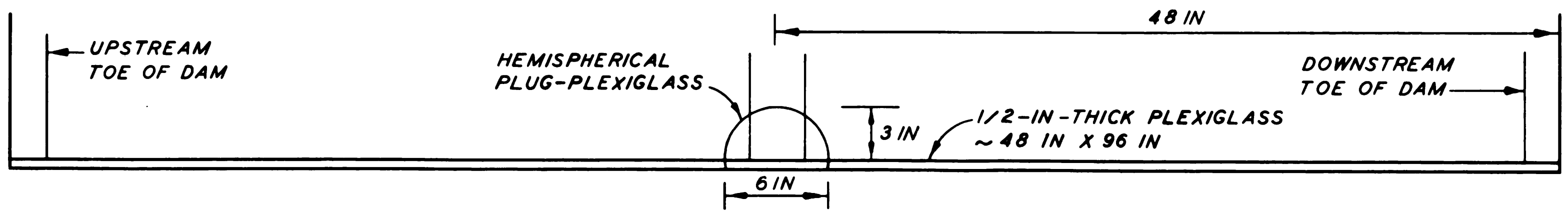
3.2.1. Model Test

In order to examine qualitatively the effect of a buried explosion on the typical earth dam section, a small-scale model was constructed. It was assumed that the buried explosion would result in formation of a spherical cavity within the impervious central core of the dam permitting a direct connection for seepage between the pervious shells. Inasmuch as formation of such a cavity would produce a three-dimensional gravity type seepage flow which requires complex analytical formulations and computer solutions, small-scale physical models offer a practical and simple means for obtaining solutions to such a problem.

As shown in Figure 3.3, a model of the typical earth dam was constructed in a tank having dimensions 96 inches in length by 48 inches in width and 15 inches in depth. One side of the tank was constructed of plexiglass in which a hemispherical solid plug was inserted during construction of the dam with provisions made for its subsequent removal to simulate formation of a cavity (see Figure 3.4). Before testing, the plug was replaced by a cylindrical insert of plexiglass having the same thickness as the tank wall (see Figure 3.5). The cavity corresponded to a prototype cavity with a 30-foot radius corresponding to a charge weight of approximately 20 tons. The model dam itself was constructed to a scale of 1 inch equals 10 feet. The upstream and downstream shells were constructed of lightly compacted fine sand. The central impervious core was constructed of a fat clay except in the vicinity of the cavity where modeling clay was used. No downstream drainage was provided downstream of the core. It was necessary to construct the model in the dry and to remove the hemispherical plug forming the cavity prior to



a. FRONT VIEW OF TANK SHOWING MODEL DAM



b. SECTION THROUGH FRONT FACE OF TANK

Figure 3.3. Dimensions of model tank and dam.

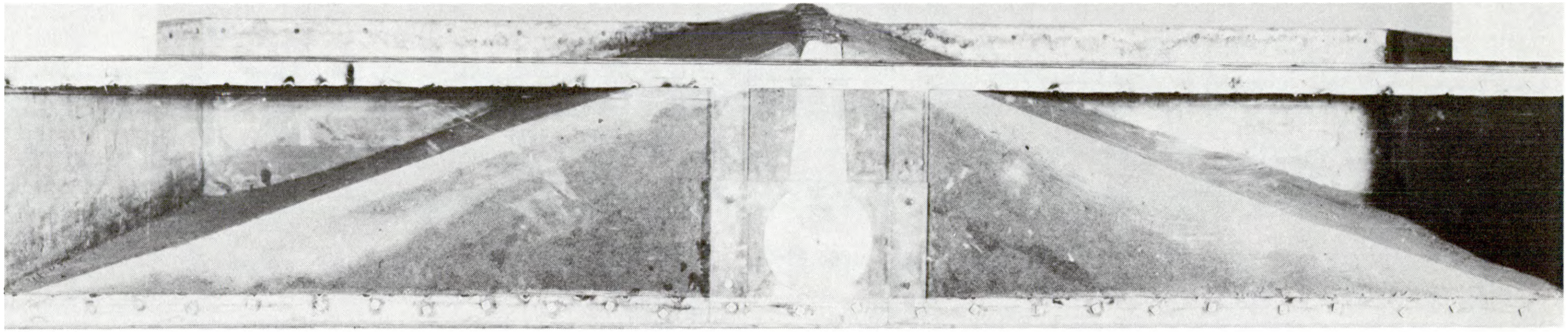


Figure 3.4. Photograph of model with hemispherical plug in place.

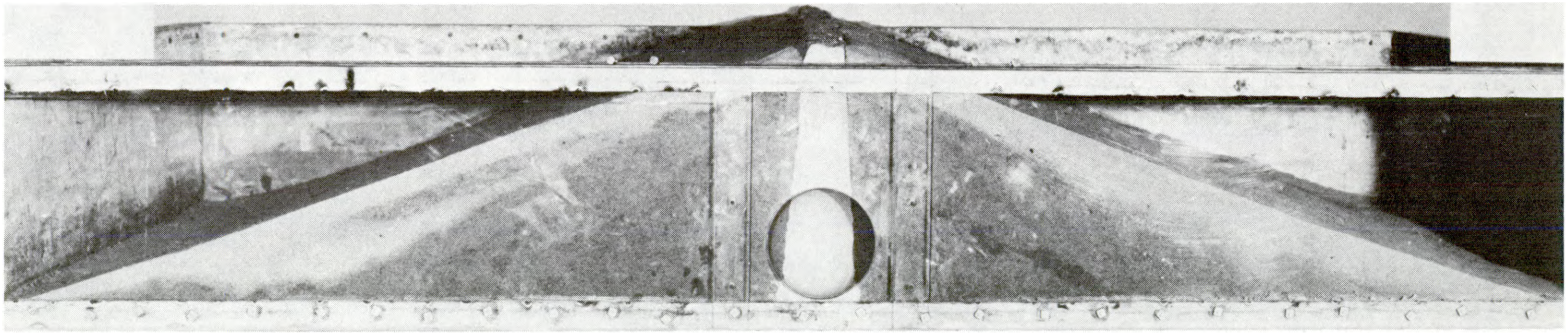


Figure 3.5. Photograph of model with hemispherical plug removed.

addition of water upstream of the dam. Otherwise, sufficient time would not have been available to replace the plug with the cylindrical insert before rapid piping would have occurred.

3.2.2. Observations

Water was allowed to enter the tank upstream of the dam and visual observations of the phenomena were made. As the reservoir approached its maximum level as shown in Figure 3.6, seepage forces caused piping of upstream shell material into the cavity which in turn caused a collapse crater upstream of the core. Seepage downstream of the core began to develop; however, because of capillary forces, the phreatic surface was not easily distinguishable. In Figure 3.7, the collapse of the cavity is essentially complete; the upstream subsidence crater is fairly well developed, and the seepage pattern downstream is also more or less fully developed. It was possible to define the approximate surface of the phreatic line downstream of the core. Because of seepage forces, the downstream slope of the embankment became relatively unstable with surface ravelling and a small unintentional shock to the tank caused the downstream slope surface to become quick and fail due to sloughing.

The observed phreatic surface in the model test downstream of the core was used to construct a flow net for the downstream portion of the dam as shown in Figure 3.8b. Also shown for comparison are flow nets for the undamaged earth dam section (Figure 3.8a) and a homogeneous earth dam section without a central core (Figure 3.8c). For the typical earth dam section with a central impervious core, seepage would be negligible, as shown in Figure 3.8b. On the other hand, the maximum seepage concentrated in the vicinity of the buried explosion is calculated from Figure 3.8b to be 16.7 k where k is the permeability of the shell materials. The seepage through the homogeneous earth dam is calculated to be approximately 12 k. Thus, it can be concluded that the buried explosion would result in a maximum seepage flow about 25 percent greater than if the dam had no central core. The amount of seepage would be dependent on the permeability of the shell materials. It is very unlikely that the typical dam would be constructed without any

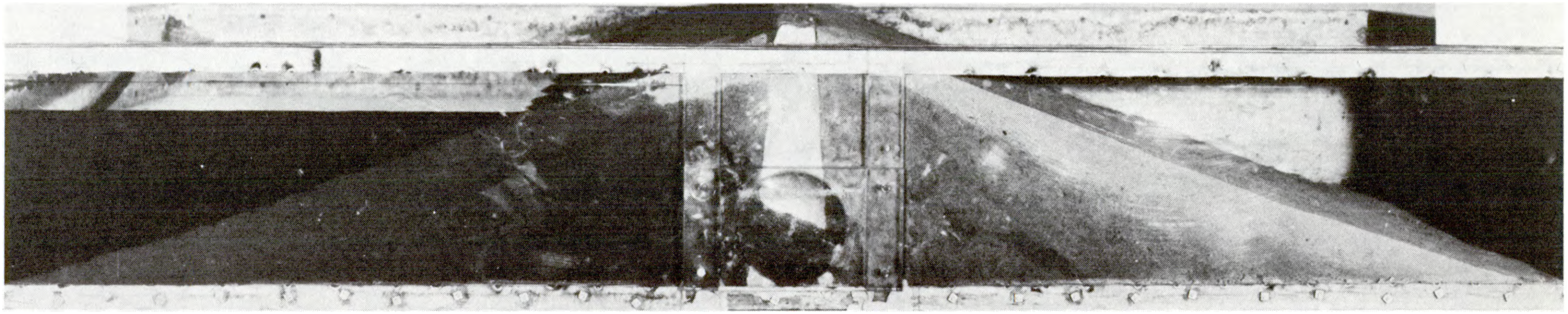


Figure 3.6. Initial collapse of cavity due to seepage.

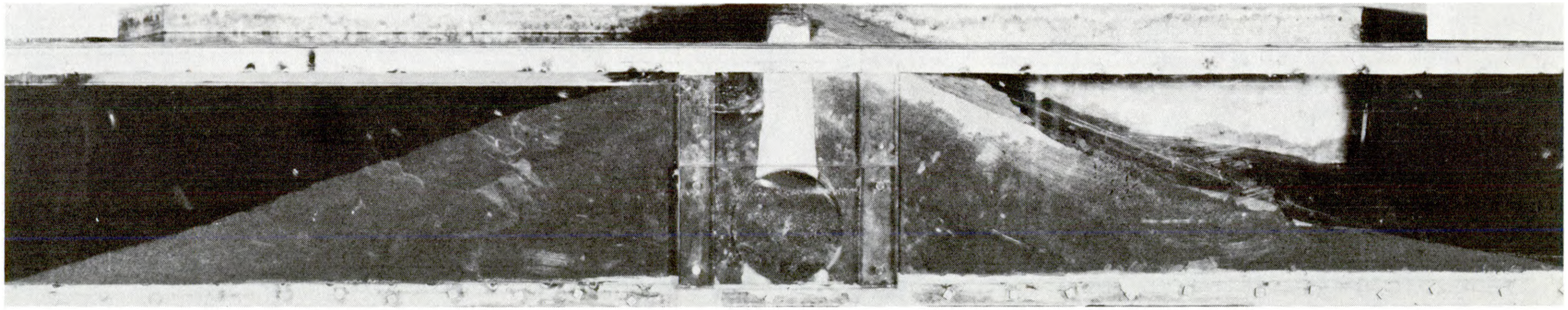


Figure 3.7. Final collapse of cavity with steady seepage.

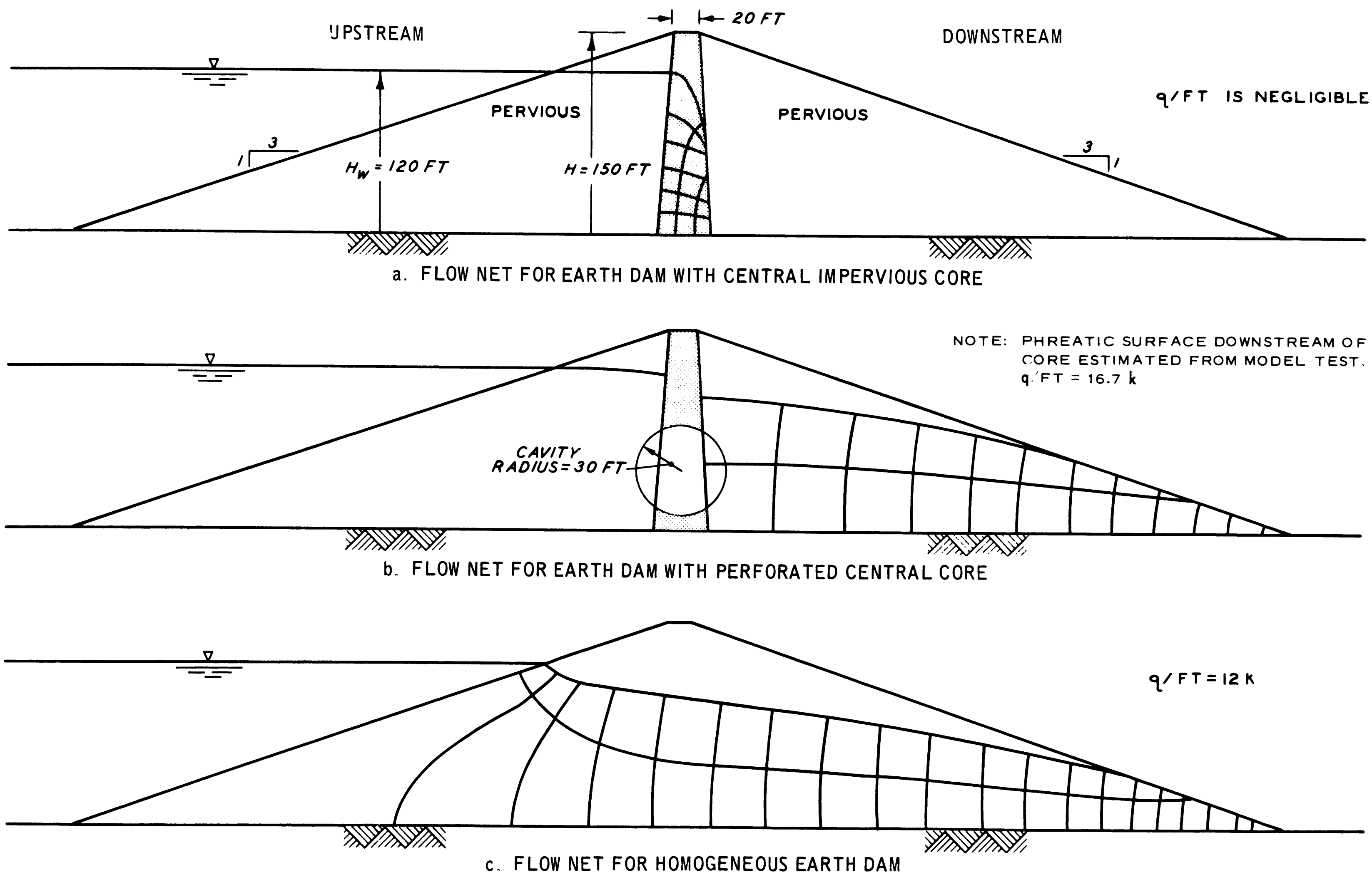


Figure 3.8. Flow nets for typical earth dam section.

provisions for downstream drainage; therefore, the increased seepage could be safely contained and handled by downstream drainage provisions.

The above analysis assumes that fissures and cracks around the cavity would have a negligible influence on the seepage pattern. Actually, if such disruptions occurred as appears likely, seepage would be substantially increased and the tendency for piping to develop would also increase. The important consideration is the piping resistance of the downstream shell materials. If these materials consisted of fine sands or silts as shown by the model test, then ravelling and gradual sloughing of the downstream slopes and failure of the damaged dam would result in a relatively short time. If the materials consisted of nondispersive clays, seepage through cracks and fissures might continue indefinitely without endangering the overall stability of the dam. In some cases, however, continued seepage through cracks has caused serious damage.[28]

CHAPTER 4

DAMAGE TO UNDERSEEPAGE CONTROL MEASURES

4.1 GENERAL CONSIDERATIONS

If the model dam were founded on pervious foundation strata, underseepage would be prevented by a vertical cutoff, or one or more types of underseepage control measures could be employed. If the pervious foundation were not too thick, extension of the impervious core to the pervious strata would provide a positive cutoff. In the case of thick pervious strata, impermeable cutoffs formed by slurry trenches, concrete walls, or grout curtains could be used. Methods for reducing and controlling underseepage are summarized in Figure 4.1. Uncontrolled underseepage may threaten the safety of a dam in two ways: (1) by reducing the stability of the downstream slope with respect to sliding or possibly causing a failure due to heave of the top stratum at the toe or (2) by causing a piping failure.

A review of the literature disclosed no major earth dams failing as result of underseepage. However, such notable failures as the Baldwin Hills Reservoir[15] have been initiated by uncontrolled underseepage in the foundation. There have been no reported failures of earth dams in which the dam failed because of uplift pressures although Marsland[29] reported the breach of a levee caused by excessive uplift pressures in a pervious foundation. High uplift pressures in pervious foundation strata may also have been a factor in the failure of downstream slopes of some dams. Some failures of Mississippi River levees have been attributed to piping caused by uncontrolled underseepage.[30] Subsurface erosion of sufficient magnitude could result in gradual or sudden subsidence of a dam to the extent that it could be overtopped.

4.2 EFFECTS OF BURIED EXPLOSIONS

Buried explosions may be used to nullify the effectiveness of underseepage control measures in various ways. Upstream blankets consisting of naturally impervious top stratum deposits or else compacted impervious borrow materials are often provided to reduce the quantity of

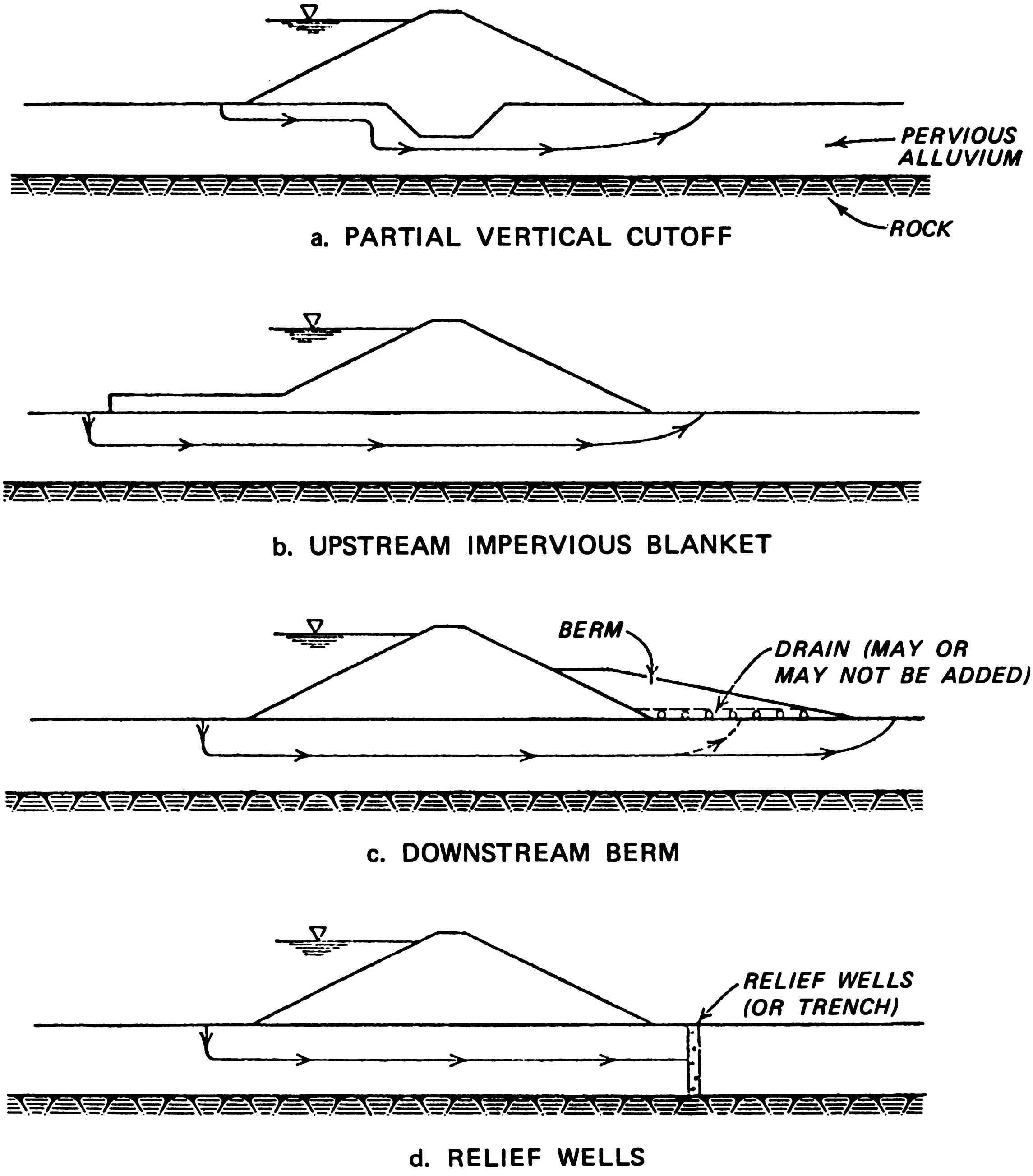


Figure 4.1. Methods of underseepage control for dams on pervious foundations without complete seepage cutoffs.

underseepage for dams constructed on pervious foundations. One possibility for increasing the amount of underseepage is to rupture the upstream blanket near the upstream toe of the dam by means of a relatively shallow buried explosion. The resulting crater would provide free access of reservoir waters into the foundation. The increased seepage would be a function of the thickness and permeability of the pervious foundation. It is expected, however, that the seepage would dissipate laterally into the foundation so that the effects at the downstream toe would be distributed over a wide distance and would probably have little effect on the stability of the dam. The increased seepage could be calculated from three-dimensional finite element analyses or by means of a three-dimensional electrical analogy model. An apparatus of this type is available at WES.

Another possibility is to use a buried explosion to perforate the impervious cutoff or barrier beneath the center of the dam. Again perforation of the barrier would allow additional seepage; however, it is expected that the effects would again be dispersed laterally and would not cause problems from the standpoint of concentrated leakage downstream of the dam. The effects, however, would be more pronounced than that arising from damage to the upstream blanket. Three-dimensional electrical analogy tests would be useful in the determination of the pattern of underseepage resulting from perforation of the barrier. A secondary effect would be large pore pressures generated by the explosion which could affect the stability of the downstream slope of the dam. At the present time, there appears to be no basis for making even an approximate estimate of what these pore pressures might be.

A buried explosion or crater formed at the downstream toe of the dam protected by relief wells would destroy some of the relief wells resulting in seepage concentrations at the toe. Relief wells are normally spaced along the toe of the dam at distances between 50 to 200 feet, and an explosion would destroy possibly only a few of the relief wells. Very seldom is reliance for underseepage control placed solely on relief wells, and the possibility of inducing large-scale piping failure would

depend on so many factors that any prediction would necessitate detailed knowledge of the specific dam.

It should be noted that if a large leak develops through relatively coarse foundation strata, it is probable that progressive backward erosion or piping would not occur. On the other hand, even a small leak concentrated at the toe of the dam could be hazardous if the foundation soils consisted of fine materials. Cohesionless silts and very fine uniform sands would be the most critical from this standpoint. In summary, a buried explosion in the foundation of the model dam would not appear to cause as much damage as detonation of a similar yield device within the embankment. The most effective location for the device and an estimation of the probable effects can be determined provided detailed knowledge of the soil conditions and design details are available.

CHAPTER 5

DOWNSTREAM SLIDES

In the case of the model dam with a central impervious core, the stability with respect to sliding is dependent primarily on the downstream shell remaining intact. Many earth dams have experienced substantial slides without affecting the overall integrity of the dam (Reference 5). Breaching would not normally occur unless the central core were involved in the slide or unless sufficient movements were generated in the core to induce extensive cracking. The obvious means of breaching would involve cratering of the crest resulting in overtopping; however, this aspect is covered under a related study. Another means by which the dam could be breached would involve removal of a substantial portion of the downstream shell by cratering so as to cause downstream displacement of the core. Buried explosions beneath the embankment slope in general would not be overly effective, as model tests (Reference 31) in cohesionless materials indicate that apparent craters below an inclined surface have a smaller volume than that produced by an explosion below a horizontal surface. The difference is illustrated in Figure 5.1 and is explained by the fact that a considerably greater portion of the material undergoes displacement and assumes a looser structure when the surface is sloped. Assuming that the model dam is founded on a rock foundation, it appears that a buried explosion within the limits of the downstream slope would probably be ineffective in removing a sufficient amount of embankment material so as to induce a downstream slide.

It should be realized that embankment dams are normally conservatively designed and, thus, relatively stable structures. A factor of safety with respect to sliding for long-term conditions is typically about 1.5. With the assumption that the model dam is founded on a clay stratum 20 feet in thickness, the required shear strength of the clay stratum to realize a factor of safety of 1.5 is calculated at 3000 psf. Thus, to produce a failure of the downstream slope under these conditions, i.e. reduce the factor of safety with respect to sliding to less

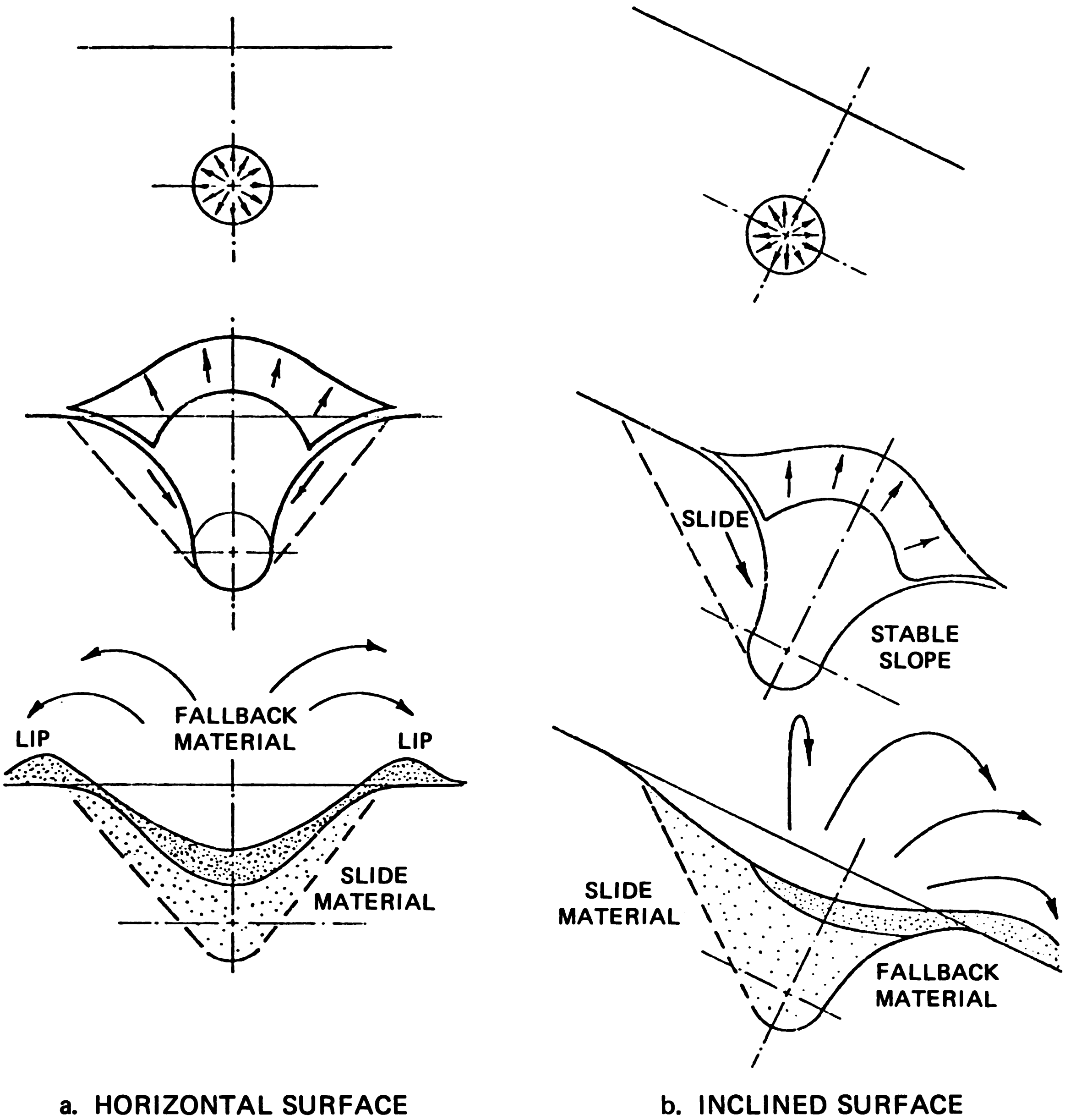


Figure 5.1. Mechanisms of cratering in a sand mass.

than unity, it is calculated that approximately 270 feet of the downstream portion of the clay stratum would have to be removed by cratering from a buried explosion. The above analysis ignores three-dimensional effects and reduction in shear strength beyond the crater due to remolding but does indicate that explosions with yields in excess of 100 tons would be required to induce a downstream slide on the model dam.

CHAPTER 6

VULNERABILITY OF OTHER TYPES OF EARTH AND EARTH ROCK DAMS

In addition to the more or less conventional earth dam design typified by the model dam, many dams are constructed either as rock-fill or hydraulic-fill dams. Some brief comments on the vulnerability of these are presented below.

6.1 ROCK-FILL DAMS

Many types of rock-fill dams, as shown in Figure 6.1, have been constructed. During the past decade, there has been a trend toward building higher dams of this type, particularly for hydroelectric purposes. Many of these are constructed as rock-fill dams, either with a thin clay core or else an impervious upstream membrane of either concrete, steel, asphalt, etc. Damages due to buried explosions would include rupturing of the thin clay core or the upstream membrane. As previously noted, the seepage through the perforation would tend to dissipate laterally and, as such, would have practically no effect on the stability of the dam.

Rather extensive model tests were performed in Norway to investigate the rate of seepage in the case of limited failure of the upstream membrane of rock-fill dams.[32] Damage was considered in terms of greater through flow than that normally classified as leakage, for instance that caused by explosives. Damage criteria were developed in terms of a factor A_r defined as the quotient between an opening A_d in the upstream impervious membrane and the total submerged surface of the membrane. A corresponding definition of a relative seepage Q_r is given as the quotient between the leakage Q_d and the seepage Q_s when the upstream membrane is missing entirely. The relation between relative seepage and the damage factor is shown in Figure 6.2. The model tests indicated that relative seepage is independent of the location of the damaged area. For damage factors less than 1 percent, the following relation was suggested:

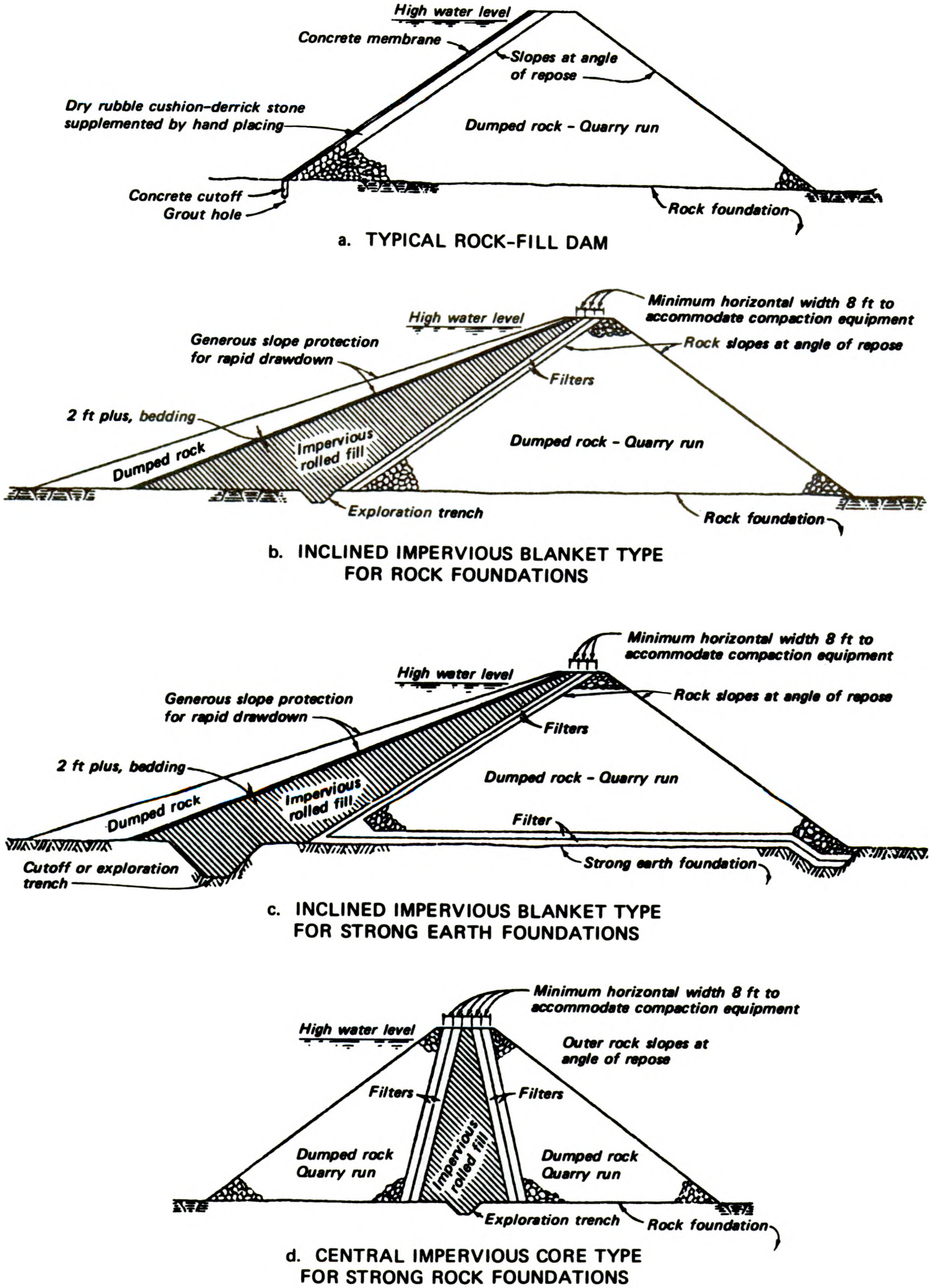


Figure 6.1. Typical rock-fill dam sections.

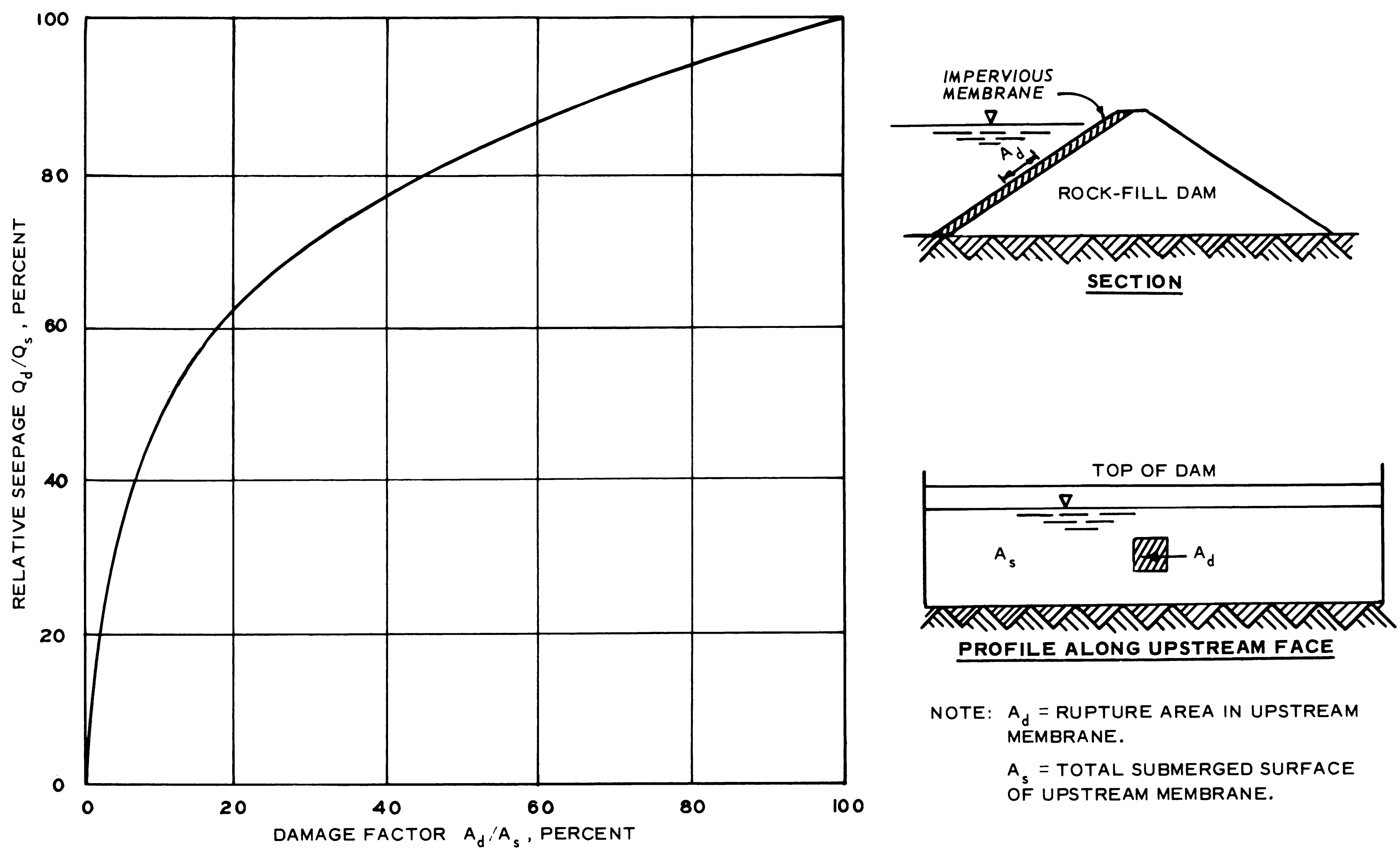


Figure 6.2. Relative seepage versus damage factor for rock-fill dams.

$$Q_r = CA_r$$

The value of C was not investigated but a value between 10 and 20 was suggested.

Perforation of the upstream membrane by a surface or buried explosion could result in a downward slide of upper portions of the membrane so that a vertical slit is opened along the entire upstream face of the dam. This would be more likely in the case of reinforced concrete slabs with vertical construction joints.

Recent innovations in rock-fill dam designs permit the embankment itself to function as an overflow section, and the normally erodible crest is protected together with a downstream face provided with an armored zone of large rocks and steel reinforcing bars. The failure at Hell Hole Dam described by Leps[16] is an example where failure occurred as a result of seepage through rock fill. The partially completed dam section, which was unreinforced, experienced small, progressively enlarging slides starting from a point of concentration of flows, which eventually breached the dam in a period of less than 4-1/2 hours under gradients less than 0.75 and probably no more than 0.3 to 0.4. Leps concluded that instability would have been unlikely to develop if removal of the larger rocks at the toe of the embankment were prevented either by their large size or by being tied back.

6.2 HYDRAULIC-FILL DAMS

The hydraulic-fill method of constructing dams was widely used during the early part of this century. The method has considerable economic advantage over other types of construction. The method was essentially abandoned in the United States after the failure of Fort Peck Dam in 1938. However, this method of construction continues to find widespread use in the Soviet Union.[33] Soviet practice differs somewhat from American practice in that material is sluiced over a surface rather than filled in ponds. The method referred to as "thin layer hydraulic filling" is used to construct dams in seismic areas. The dams are

checked for possibility of liquefaction, although the procedures for making such checks are not known.

Experience indicates that under earthquake loading, hydraulic-fill dams may be susceptible to liquefaction. The tendency to liquify depends on the gradation and relative density of the shell materials. Most dams of this type are constructed with relatively flat slopes. Results of blasting tests in loose sands suggest that residual pore pressures may be generated for substantial distances away from the blast point. Consequently, it appears that hydraulic-fill dams are likely to fail due to liquefaction as a result of a buried explosion. However, no direct experience is available, and prediction of the type and nature of liquefaction failure is an exceedingly complex subject at the present time. Centrifuge modeling may offer a means for assessing the degree of damage induced by buried explosions in hydraulic-fill dams.

CHAPTER 7

CONCLUSIONS

Available data are insufficient to estimate reliably the size and characteristics of camouflet that would be formed by a buried explosion. Practically nothing is known concerning the nature of the rupture zone and extent of cracking which might develop in both saturated and unsaturated materials as might be involved in an earth dam. The magnitude and distribution of peak and residual pore pressures which would develop are unknown. Further experimental data are needed in these areas.

In the case of a model dam selected for the study, it is considered quite probable that detonation of a 20-ton buried explosion located so as to perforate the central core will cause sufficient cracking and internal erosion, depending on the nature of the embankment materials, as to result in breaching of the dam. A small-scale model test indicated that a subsidence crater would form upstream of the core after detonation and that seepage would be somewhat greater than that for a homogeneous dam. Breaching of the dam, should it occur, would probably take place in less than 5 hours once significant seepage water appeared on the downstream embankment. Buried explosions in the foundation of the dam or beneath the downstream slope would be less effective in producing failure than explosions of similar yield within the embankment itself.

Although the subject study concerns the effects of a buried explosion on a typical earth dam, it must be emphasized that every dam is unique in cross section, and the embankment slopes, zonation, and internal drainage details are tailored to the specific site conditions, the materials available for construction, and other factors. The behavior of a dam cannot always be reliably predicted during its design life where all of the details are reasonably known during the design of the structure. It is much more difficult to assess the probable behavior of a dam under the action of a buried explosion for which practically no experience is available. Small-scale model tests, particularly if tested in a centrifuge, appear to offer the best means for more detailed study. Three-dimensional electrical analogy tests would also

provide useful information. An exceedingly important question concerns the nature and extent of cracking generated by the blast in various soil types and the effects of cracking on possible piping failure. The consequences of cracking and methods for its control are subjects of much current interest to designers of earth dams. Development of a laboratory research program to evaluate piping resistance of cracked embankment material would be worthwhile in this regard.

REFERENCES

1. T. A. Middlebrooks; "Progress in Earth Dam Design and Construction in the United States"; Civil Engineering, September 1952.

2. S. I. Goldsmith; "A Study of Earth and Rockfill Dam Failures, 1950-1972"; August 1972; Master of Science Thesis, Virginia Polytechnic Institute, Blacksburg, Va.

3. A. O. Babb and T. W. Mermel; "Catalogue of Dam Disasters, Failures, and Accidents"; 1968; U. S. Department of Interior, Bureau of Reclamation.

4. International Commission on Large Dams; "Lessons from Dam Incidents"; Reduced Edition; 1973.

5. United States Committee on Large Dams; "Lessons from Dam Incidents, USA"; 1975; published jointly by the American Society of Civil Engineers and the United States Committee on Large Dams.

6. A. C. Casagrande; "Notes on the Design of Earth Dams"; Journal of the Boston Society of Civil Engineers; October 1950; Vol. 37, No. 4.

7. J. L. Sherard and others; "Earth and Earth-Rock Dams"; 1963; John Wiley & Sons, Inc., New York.

8. U. S. Army Corps of Engineers; "Earth and Rock-fill Dams - General Design and Construction Considerations"; EM 1110-2-2300; March 1971.

9. E. Waelter; "Determination of the Origin and Quantity of the Leakage and Its Influence on the Behavior of the Sorpe Dam"; Transactions of the Twelfth International Congress on Large Dams; 1976; Mexico.

10. A. D. Rooke, B. L. Carnes, and L. K. Davis; "Cratering by Explosions: A Compendium and an Analysis"; Technical Report TR N-74-1, January 1974; U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg Miss.

11. U. S. Army Engineer Waterways Experiment Station, CE; "Cratering Effects of Surface and Buried HE Charges in Loess and Clay"; Technical Report TR 2-482, June 1958.

12. S. J. Johnson; "A Limited Review of Seismic Design of Embankment Dams Constructed over Potentially Active Faults"; (Unpublished Report), January 1976; U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

13. H. P. Cedergren; "Seepage, Drainage, and Flow Nets"; 1968; John Wiley & Sons, Inc., New York.

14. E. B. Perry; "Piping in Earth Dams Constructed of Dispersive Clay; Literature Review and Design of Laboratory Tests"; Technical Report TR S-75-15, November 1975; U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

15. A. C. Casagrande, S. D. Wilson, and W. D. Schwartes; "The Baldwin Hills Reservoir Failure in Retrospect"; ASCE Specialty Conference on Earth and Earth-Supported Structures; June 1972; Purdue University, Lafayette, Ind.

16. T. M. Leps; "Flow Through Rockfill"; A Chapter in Embankment-Dam Engineering, Casagrande Volume, 1973; John Wiley & Sons, Inc., New York.

17. "Wyoming Dam Fails"; Engineering News-Record, 17 July 1969.

18. S. J. Johnson; "Failure of Walter Bouldin Dam"; Memorandum for Record, 24 April 1975; U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

19. W. E. Davies, J. F. Bailey, and D. B. Kelly; "West Virginia's Buffalo Creek Flood: A Study of the Hydrology and Engineering Geology"; Geological Survey Circular 667, 1972; Washington.

20. R. J. Marsal and W. Pohlenz; "The Failure of Laguna Dam"; ASCE Specialty Conference on Performance of Earth and Earth-Supported Structures, June 1972; Purdue University, Lafayette, Ind.

21. E. R. Tinney and H. Y. Hsu; "Mechanics of an Erodible Fuse Plug"; Transactions, American Society of Civil Engineers, 1962; Vol. 127.

22. J. L. Sherard; "Embankment Dam Cracking"; A Chapter in Embankment-Dam Engineering, Casagrande Volume, 1973; John Wiley & Sons, Inc., New York.

23. U. S. Department of the Interior; "Interim Report of the Teton Dam Failure"; 14 July 1976; Teton Dam Failure Review Group.

24 J. L. Sherard; "Influence of Soil Properties and Construction Methods on the Performance of Homogeneous Earth Dams"; Bureau of Reclamation Technical Memorandum 645, January 1953.

25. W. C. Sherman; "Information on Damage and Failures of Dams in California"; Memorandum for Record, 20 July 1976; U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

26. J. L. Sherard, R. S. Decker, and N. L. Ryker; "Piping in Earth Dams of Dispersive Clay"; ASCE Specialty Conference on Performance of Earth and Earth-Supported Structures, 11-14 June 1972; Purdue University, Lafayette, Ind.

27. J. L. Sherard, L. S. Cluff, and C. R. Allen; "Potentially Active Faults in Dam Foundations"; Geotechnique, Vol. 24, No. 3, September 1976.

28. P. R. Vaughn and others; "Cracking and Erosion of Balderhead Dam and the Remedial Works Adopted for Its Repair"; 10th Congress on Large Dams, Vol. 1, 1970; Montreal.

29. A. Marsland; "A Study of a Breach in an Earthen Embankment Caused by Uplift Pressures"; Proceedings of the Fifth ICSMFE, Vol. II, 1961; Paris.

30. U. S. Army Engineer Waterways Experiment Station, CE; "Investigation of Underseepage and Its Control, Lower Mississippi River Levees"; Technical Memorandum 3-424, October 1956.

31. A. S. Vesic and Tien-Lie Tai; "Theoretical Studies of Cratering Mechanisms Affecting the Stability of Cratered Slopes"; Report 6, TR 3-699, March 1967; U. S. Army Engineer Waterways Experiment Station, CE, Vicksburg, Miss.

32. O. Solvik and R. Svee; "Flow Condition Criteria and Some Throughflow Problems in Rockfill"; Transactions of the Twelfth International Congress on Large Dams, 1976; Mexico.

33. N. N. Yakaulev; "Some Data About Earth and Rockfill Dams in the U.S.S.R."; Irrigation and Power, July 1967.

DISTRIBUTION LIST

Address	No. of Copies
DOD	
Assistant to the Secretary of Defense, Atomic Energy,	
Washington, D. C. 20301	
ATTN: Donald R. Cotter	1
Director, Defense Advanced Research Projects Agency, Architect	
Building, 1400 Wilson Blvd., Arlington, Virginia 22209	
ATTN: Technical Library	1
Director, Defense Civil Preparedness Agency,	
Washington, D. C. 20301	
ATTN: RE(ADM)	1
Defense Documentation Center, Cameron Station, Alexandria,	
Virginia 22314	
ATTN: TC/Myer B. Kahn	2
Director, Defense Intelligence Agency, Washington, D. C. 20301	
ATTN: DT-2, Weapons & Systems Division	1
DI-7D, Phys. Vul. Div., Edward O'Farrell	1
Technical Library	1
Director, Defense Nuclear Agency, Washington, D. C. 20305	
ATTN: STSI (Archives)	1
STTL (Technical Library)	2
SPSS	2
DDST	1
SPSS/Tom Kennedy	1
Director of Defense Research and Engineering,	
Washington, D. C. 20301	
ATTN: Dep Dir, Strategic Systems	1
Dep Dir, Tactical Warfare	1
Interservice Nuclear Weapons School, Kirtland Air Force Base,	
New Mexico 87115	
ATTN: Technical Library	1
Chairman, Department of Defense Explosives Safety	
Board, Room 6A145, Forrestal Building,	
Washington, D. C. 20314	
ATTN: Dr. Tom Zaker	1

Address	No. of Copies
DOD (Continued)	
Director, Joint Strategic Target Planning Staff, JCS, Offutt Air Force Base, Nebraska 68113	
ATTN: Science and Technology Info Library	1
Commander, Field Command, Defense Nuclear Agency, Lawrence Livermore Laboratory, P. O. Box 808, Livermore, California 94550	
ATTN: FCTA-D	1
Director, Weapons Systems Evaluation Group, 400 Army-Navy Drive, Arlington, Virginia 22202	
ATTN: Technical Library	1
Army	
Commander, U. S. Army Ballistic Missile Defense Systems Command, P. O. Box 1500, Huntsville, Alabama 35807	
ATTN: BMDSC-TEN (Noah J. Hurst)	1
BMDSC-HS	1
Office, Chief of Engineers, Department of the Army, Washington, D. C. 20314	
ATTN: DAEN-ASI-L	2
DAEN-MCE-D	1
DAEN-RDL	1
DAER-RDM	1
Engineer Studies Group, 6500 Brooks Lane, Washington, D. C. 20315	
ATTN: DAEN-FES/George H. Orrell	1
Commander, Harry Diamond Laboratories, 2800 Powder Mill Road, Adelphi, Maryland 20783	
ATTN: AMXDO-NP	1
AMXDO-TI, Technical Library	1
Office, Chief of Research, Development, and Acquisition, Department of the Army, Washington, D. C. 20310	
ATTN: DAMA-CSM	1
Technical Library	1
Deputy Chief of Staff for Operations and Plans Department of the Army Washington, D. C. 20310	
ATTN: Director of Chemical and Nuclear Operations	1
Technical Library	1

Address	No. of Copies

Army (Continued)

Commanding Officer, Picatinny Arsenal, Dover, New Jersey 07801	
ATTN: P. Angelloti	1
Technical Library	1
SMUPA-ND-S/E. Zimpo	1
Director, U. S. Army Ballistic Research Laboratories, Aberdeen Proving Ground, Maryland 21005	
ATTN: AMXBR-X/J. J. Meszaros	1
J. H. Keefer	1
W. J. Taylor	1
Technical Library/Edward Baicy	2
Commandant, U. S. Army Engineer School, Fort Belvoir, Virginia 22060	
ATTN: ATSEN-SY-L	1
Division Engineer, U. S. Army Engineer Division, Huntsville, P. O. Box 1600, West Station, Huntsville, Alabama 35807	
ATTN: HNDED-CS/Michael M. Dembo	1
HNDED-SR	1
Division Engineer, U. S. Army Engineer Division, Ohio River, P. O. Box 1159, Cincinnati, Ohio 45201	
ATTN: ORDAS-L/Technical Library	1
Director, U. S. Army Engineer Waterways Experiment Station, P. O. Box 631, Vicksburg, Mississippi 39180	
ATTN: J. W. Brown	1
L. F. Ingram	1
William J. Flathau	1
John Strange	1
Library	5
Commander, U. S. Army Materials and Mechanics Research Center, Watertown, Massachusetts 02172	
ATTN: Technical Library	1
Commander, U. S. Army Materiel Development and Readiness Command, (DARCOM) 5001 Eisenhower Avenue, Alexandria, Virginia 22333	
ATTN: Technical Library	1
DRCRD-WN	2

Address	No. of Copies

Army (Continued)

Commander, U. S. Army Missile Command, Redstone Arsenal, Alabama 35809	
ATTN: Technical Library	1
AMCPM-PE-X (William K. Jann)	1
W. Fowler/AMSMI-YDR	1
Commander, U. S. Army Missile Command, Redstone Scientific Information Center, Redstone Arsenal, Alabama 35809	
ATTN: Chief, Documents	1
Commander, U. S. Army Communications Command Fort Huachuca, Arizona 85613	
ATTN: Technical Library	1
Commander, U. S. Army Mobility Equipment Research and Development Center, Fort Belvoir, Virginia 22060	
ATTN: Technical Library	1
Commander, U. S. Army Nuclear Agency, 7500 Backlick Road, Bldg 2073, Springfield, Virginia 22150	
ATTN: CDINS-E	1
Technical Library	1
Commander, U. S. Army Nuclear Agency Fort Bliss, Texas 79916	
ATTN: Technical Library	1
ATCA-NAW	1
Commander, U. S. Army Armament Command, Rock Island Arsenal, Rock Island, Illinois 61201	
ATTN: Technical Library	1
Commander, U. S. Army Tank Automotive Command Warren, Michigan	
ATTN: Technical Library	1

Navy

Chief of Naval Operations, Department of the Navy, Washington, D. C. 20350	
ATTN: OP-985F	1
Chief of Naval Research, Department of the Navy, Arlington, Virginia 22217	
ATTN: Technical Library	1

Address	No. of Copies
Navy (Continued)	
Officer in Charge, Civil Engineering Laboratory, Naval Construction Battalion Center, Port Hueneme, California 93043	
ATTN: Mr. R. J. Odello	1
Technical Library	1
Commander, Naval Electronics Systems Command, Headquarters, Washington, D. C. 20360	
ATTN: PME-117-21A	1
Commander, Naval Facilities Engineering Command, Headquarters Washington, D. C. 20390	
ATTN: Code 03A	1
Technical Library	1
Commander, Naval Facilities Engineering Command, Hoffman Building, 200 Stovall Street, Alexandria, Virginia 22332	
ATTN: Technical Library, Code 0911C	1
Superintendent, U. S. Naval Postgraduate School, Monterey, California 93940	
ATTN: Library, Code 2124	1
Director, Naval Research Laboratory, Washington, D. C. 20375	
ATTN: Library, Code 2029	1
Technical Library, Code 2027	1
Commander, Naval Ship Research and Development Center, Underwater Explosive Research Division, Portsmouth, Virginia 23709	
ATTN: Technical Librarian	1
Commander, Naval Surface Weapons Center, Dahlgren Laboratory, Dahlgren, Virginia 22448	
ATTN: Technical Library	1
Commander, Naval Surface Weapons Center, White Oak, Silver Spring, Maryland 20910	
ATTN: Technical Library, Code 730	1
Code 1224 Navy Nuc Prgms Off	1
Code WA501 Navy Nuc Prgms Off	1
Commander, Naval Weapons Center, China Lake, California 93555	
ATTN: Code 533, Technical Library	1

Address	No. of Copies
Navy (Continued)	
Commander, Naval Weapons Evaluation Facility, Kirtland AFB, Albuquerque, New Mexico 87117	
ATTN: Technical Library	1
R. Hughes	1
Director, Strategic Systems Project Office, Navy Department, Washington, D. C. 20376	
ATTN: NSP-43, Technical Library	1
Air Force	
Air Force Armament and Testing Laboratory, AFSC, Eglin Air Force Base, Florida 32542	
ATTN: Technical Library	1
Air Force Cambridge Research Laboratories, AFSC, L. G. Hanscom Field, Bedford, Massachusetts 01730	
ATTN: LWW, Dr. Ker Thompson	1
SUOL AFCRL, Research Library	1
Air Force Institute of Technology, AFIT Building 640, Area B, Wright-Patterson Air Force Base, Ohio 45433	
ATTN: Technical Library	1
Commander, Air Force Weapons Laboratory, AFSC, Kirtland Air Force Base, New Mexico 87117	
ATTN: DEV-G/Mr. R. W. Henny	1
SUL, Technical Library	1
DEV-S/Dr. M. A. Plamondon	1
Commander, Field Command, Defense Nuclear Agency Kirtland Air Force Base, New Mexico 87115	
ATTN: FCPR	1
Headquarters, Air Force Systems Command, Andrews Air Force Base, Washington, D. C. 20331	
ATTN: Technical Library	1
Air Force Geophysics Laboratory, Air Force Systems Command Hanscom Air Force Base, Maine 01731	
ATTN: SUOL AFCRL Research Library	1
Air Force Weapons Laboratory, Air Force Systems Command Kirtland Air Force Base, New Mexico 87117	
ATTN: Robert Port	1

Address	No. of Copies
Air Force (Continued)	
Commander, Armament Development and Test Center, Eglin Air Force Base, Florida 32542	
ATTN: ADTC/DLODL/Technical Library	1
Commander, Foreign Technology Division, AFSC, Wright-Patterson Air Force Base, Ohio 45433	
ATTN: TD-BTA Library	1
Commander, Rome Air Development Center, AFSC, Griffiss Air Force Base, New York 13440	
ATTN: EMTLD, Documents Library	1
Space and Missile Systems Organization, Norton Air Force Base, California 92409	
ATTN: MMH/Hard Rock Silo Development	1
MMH/Engineering Division	1
DEB (Civil Engineering)	1
Space and Missile Systems, Norton Air Force Base, California 92409	
ATTN: MMH (Minuteman)	1
MNNH (Minuteman)	1
Commander, Strategic Air Command, Offutt Air Force Base, Nebraska 68113	
ATTN: NRI-STINFO Library	1
U. S. Energy Research and Development Administration	
Los Alamos Scientific Laboratory, P. O. Box 1663, Los Alamos, New Mexico 87544	
ATTN: Document Control for Reports Library	1
Sandia Laboratories, Livermore Laboratory, P. O. Box 969, Livermore, California 94550	
ATTN: Document Control for Technical Library	1
Sandia Laboratories, P. O. Box 5800, Albuquerque, New Mexico 87115	
ATTN: Document Control for L. J. Vortman	1
Document Control for 3141 Sandia Rpt Coll	1
U. S. Energy Research and Development Administration, Albuquerque Operations Office, P. O. Box 5400, Albuquerque, New Mexico 87115	
ATTN: Document Control for Technical Library	1

Address	No. of Copies
ER&DA (Continued)	
U. S. Energy Research and Development Administration, Nevada Operations Office, P. O. Box 14100, Las Vegas, Nevada 89114	
ATTN: Document Control for Technical Library	1
U. S. Energy Research and Development Administration, Division of Headquarters Services, Library Branch G-043, Washington, D. C. 20545	
ATTN: Doc Control for Class. Tech Lib	1
Director, Lawrence Livermore Laboratory, P. O. Box 808, Livermore, California 94550	
ATTN: Technical Library	1
Larry Woodruff, L-125	1
Dr. Al Schiff, Z Division	1
Chief, Livermore Division Field Command DNA P. O. Box 808, Livermire, California 94550	
ATTN: FCPRL	1
DOD Contractors	
Aerospace Corporation, P. O. Box 92957, Los Angeles, California 90009	
ATTN: Technical Information Services	1
Dr. Prem N. Mathur	1
Agbabian Associates, 250 North Nash Street, El Segundo, California 90245	
ATTN: Dr. M. S. Agbabian	1
Applied Theory Incorporated, 1010 Westwood Boulevard, Los Angeles, California 90024	
ATTN: Dr. John G. Trulio	2
Battelle Memorial Institute 505 King Avenue, Columbus, Ohio 43201	
ATTN: R. W. Klingesmith	1
Bell Telephone Laboratories, Inc., Mountain Avenue, Murray Hill, New Jersey 07974	
ATTN: Technical Report Center	1
Boeing Company, P. O. Box 3707, Seattle, Washington 98124	
ATTN: Mr. R. H. Carlson	1

Address	No. of Copies
DOD Contractors (Continued)	
Braddock, Dunn, & McDonald, Inc., P. O. Box 9274, Albuquerque International, Albuquerque, New Mexico 87119	
ATTN: Mr. Richard Hensley	1
California Research and Technology, Inc., 6269 Variel Ave., Woodland Hills, California 91364	
ATTN: Dr. K. N. Kreyenhagen	1
Technical Library	1
Electromechanical Systems of New Mexico, Inc., P. O. Box 11730, Albuquerque, New Mexico 87112	
ATTN: R. A. Shunk	1
General American Transportation Corporation, General American Research Division, 7449 N. Natchez Ave., Niles, Illinois 60648	
ATTN: Dr. G. L. Neidhardt	1
General Electric Company, TEMPO-Center for Advanced Studies, 816 State Street, Santa Barbara, California 93102	
ATTN: DASIAC	1
IIT Research Institute, 10 West 35th Street, Chicago, Illinois 60616	
ATTN: Technical Library	1
Institute for Defense Analyses, 400 Army-Navy Drive, Arlington, Virginia 22202	
ATTN: IDA Librarian Ruth S. Smith	1
Consulting and Special Engineering Services, Inc., P. O. Box 1206, Redlands, California 92373	
ATTN: Dr. J. L. Merritt	1
Technical Library	1
Kaman Avidyne, Division of Kaman Sciences Corporation, 83 Second Avenue, Northwest Industrial Park, Burlington, Massachusetts 01803	
ATTN: Technical Library	1
E. S. Crisicone	1
N. P. Hobbs	1
Kaman Sciences Corporation, P. O. Box 7463, Colorado Springs, Colorado 80933	
ATTN: Gunning Butler, Jr.	1
Technical Library	1

Address	No. of Copies

DOD Contractors (Continued)

Lockheed Missiles and Space Company, Inc., P. O. Box 504, Sunnyvale, California 94088	
ATTN: Technical Library	1
Martin Marietta Aerospace, Orlando Division, P. O. Box 5837, Orlando, Florida 32805	
ATTN: Gerbert E. McQuaig (MP-81)	1
Al Cowan	
Merritt Cases, Incorporated, P. O. Box 1206 Redlands, California 92373	
ATTN: Technical Library	1
J. L. Merritt	1
University of Illinois, 1211 Civil Engineering Building, Urbana, Illinois 61801	
ATTN: Dr. Nathan M. Newmark	1
Physics International Company, 2700 Merced St., San Leandro, California 94577	
Doc Control for Mr. Fred M. Sauer	1
Doc Control for Dr. Robert Swift	1
Technical Library	1
Doc Control for Mr. Dennis L. Orphal	1
Doc Control for Mr. Larry Behrman	1
R&D Associates, P. O. Box 9695, Marina Del Rey, California 90291	
ATTN: Dr. Harold L. Brode	1
Dr. H. F. Cooper, Jr.	1
Technical Library	1
Mr. William B. Wright	1
Dr. C. P. Knowles	1
Mr. J. G. Lewis	1
Mr. R. J. Port	1
Science Applications, Inc., Suite 216, 2201 San Pedro NE Albuquerque, New Mexico 87110	
ATTN: Mr. J. L. Bratton	1
Science Applications, Inc., 1911 North Fort Myer Drive, Suite 808, Arlington, Virginia 22209	
ATTN: Dr. W. M. Layson	1

Address	No. of Copies
DOD Contractors (Continued)	
Science Applications, Inc., 8400 Westpark Drive, McLean, Virginia 22101	
ATTN: Dr. Michael D. McDonnell	1
Science Applications, Inc., P. O. Box 2351, La Jolla, California 92037	
ATTN: Dr. Cecil Ivan Hudson, Jr.	1
Technical Library	1
Southwest Research Institute, P. O. Drawer 28510, San Antonio, Texas 78284	
ATTN: Mr. A. B. Wenzel	1
Stanford Research Institute, 333 Ravenswood Ave., Menlo Park, California 94025	
ATTN: Dr. G. Abrahamson	1
SRI Library Room G021	1
Larry Inman	1
Burt R. Gasten	1
Systems, Science and Software, P. O. Box 1620, La Jolla, California 92037	
ATTN: Dr. Donald R. Grine	1
Dr. T. D. Riney	1
Technical Library	1
The Atrium, 277 S. Washington St., Suite 202, Alexandria, Virginia 22314	
ATTN: Jack Cane	1
Teledyne Brown Engineering Company, Inc., Cummings Research Park, Huntsville, Alabama 35807	
ATTN: Dr. M. C. Patel	1
Terra Tek, Inc., University Research Park, 420 Wakara Way, Salt Lake City, Utah 84108	
ATTN: Mr. S. J. Green	1
TRW Systems Group, San Bernardino Operations, P. O. Box 1310, San Bernardino, California 92402	
ATTN: Greg Hulcher	1
E. Y. Wong 527/712	1

Address	No. of Copies
DOD Contractors (Continued)	
TRW Defense and Space Systems Group, One Space Park, Building R1/2170 Redondo Beach, California 90278	
ATTN: Dr. J. J. Ferrell	1
Technical Information Center/S-1930	1
Peter K. Dai, R1/2170	1
Weidlinger Associates, Consulting Engineers, 110 East 59th Street, New York, New York 10022	
ATTN: Dr. Melvin L. Baron	1
Weidlinger Associates, Consulting Engineers, 3000 Sand Hill Road, Suite 245, Menlo Park, California 99025	
ATTN: Dr. J. Isenberg	1

In accordance with letter from DAEN-RDC, DAEN-ASI dated 22 July 1977, Subject: Facsimile Catalog Cards for Laboratory Technical Publications, a facsimile catalog card in Library of Congress MARC format is reproduced below.

Sherman, Walter Charles

Damage to dams from nuclear weapons effects (formerly ESSEX II); vulnerability of an earth dam to buried explosions, preliminary studies / by Walter C. Sherman, Jr. Vicksburg, Miss. : U. S. Waterways Experiment Station, 1978.

60 p. : ill. ; 27 cm. (Technical report - U. S. Army Engineer Waterways Experiment Station ; S-78-3)

Prepared for Defense Nuclear Agency, Washington, D. C., under DNA Subtask L19GAXYX979, Work Unit 03, "Erosion Failure Studies."

References: p. 46-48.

1. Dam failures. 2. Dam foundations. 3. Earth dams. 4. Earth dam seepage. 5. Nuclear weapons effects. 6. Sliding. 7. Slope failures. 8. Underground explosions. 9. Vulnerability.
I. Defense Nuclear Agency. II. Series: United States. Waterways Experiment Station, Vicksburg, Miss. Technical report ; S-78-3.
TA7.W34 no.S-78-3